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PERFORMANCE OF PAVEMENT TEST SECTIONS IN THE REHABILITATED AASHO TEST ROAD

By

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16. Abstract

The performance of the original portland cement concrete pavement sections from the AASHO Test Road has been studied. Findings indicate that the original AASHO performance equation for rigid pavements predicted a greater service life expectation for the thicker pavement slabs than was being observed. To agree with the observed performance, a modified performance equation that slightly increased the performance expectation for the pavement sections 8.0 in. (203 mm) thick and reduced the expectation for the pavement sections 9.5 in. (241 mm), 11.0 in. (279 mm) and 12.5 in. (318 mm) thick was developed. Stabilized aggregate subbases, especially the BAM, improved rigid pavement performance. On the BAM subbase, rigid pavement sections developed fewer major cracks, and pavements with joints spaced at 100 ft (30.5 m) had the most uniform winter joint opening. Dowel bar corrosion was the primary cause of joint lockup in the rigid pavement test sections.

Stabilized aggregate mixtures, used as the base course in flexible pavements, had variable effects on pavement behavior. On the BAM base, the flexible pavements were smoothest. On the CAM base, transverse cracks formed which "tented" in freezing weather. On the granular base, area cracking was the primary form of pavement distress and the deepest ruts formed. Rut formation was reduced on the rigid base and the least rutting developed where asbestos fiber filler was used in lieu of part of the filler.

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INTRODUCTION

A continuing study of the behavior of a number of experimental pavement sections in the AASHO Test Road (Figure 1) has been carried out as a necessary sequel to the original test (1). These sections, together with new pavement sections that were added to the experimentation in 1962, have been studied under regular mixed highway traffic in the Road Test environment. Since the opening of the rehabilitated test road to regular highway traffic in November 1962, over 25 million vehicles of all types have traversed the experimental pavements, and the data needed to verify the Road Test equations and to draw comparisons with the new pavement sections have been collected and analyzed regularly.

Many of the test sections in the original test road were so far below the structural requirements for interstate highway service that early failure could be expected. In rehabilitating the test road as an interstate highway, these sections were replaced with new pavement sections that were more adequate. New experimental pavement sections also were built to connect the mainline test tangents into a continuous roadway through the test facility. Some of the new pavements were built as duplicates of original designs, but most represented new designs that had not been included in the original test.

The new designs incorporated bituminous aggregate mixtures (BAM) and cement aggregate mixtures (CAM) as subbases in the rigid pavement experiment and BAM, CAM, and plain PCC bases in the flexible pavement experiment. All of the original flexible pavement sections were resurfaced and two new flexible test sections were built as duplicates of original designs together with a number of

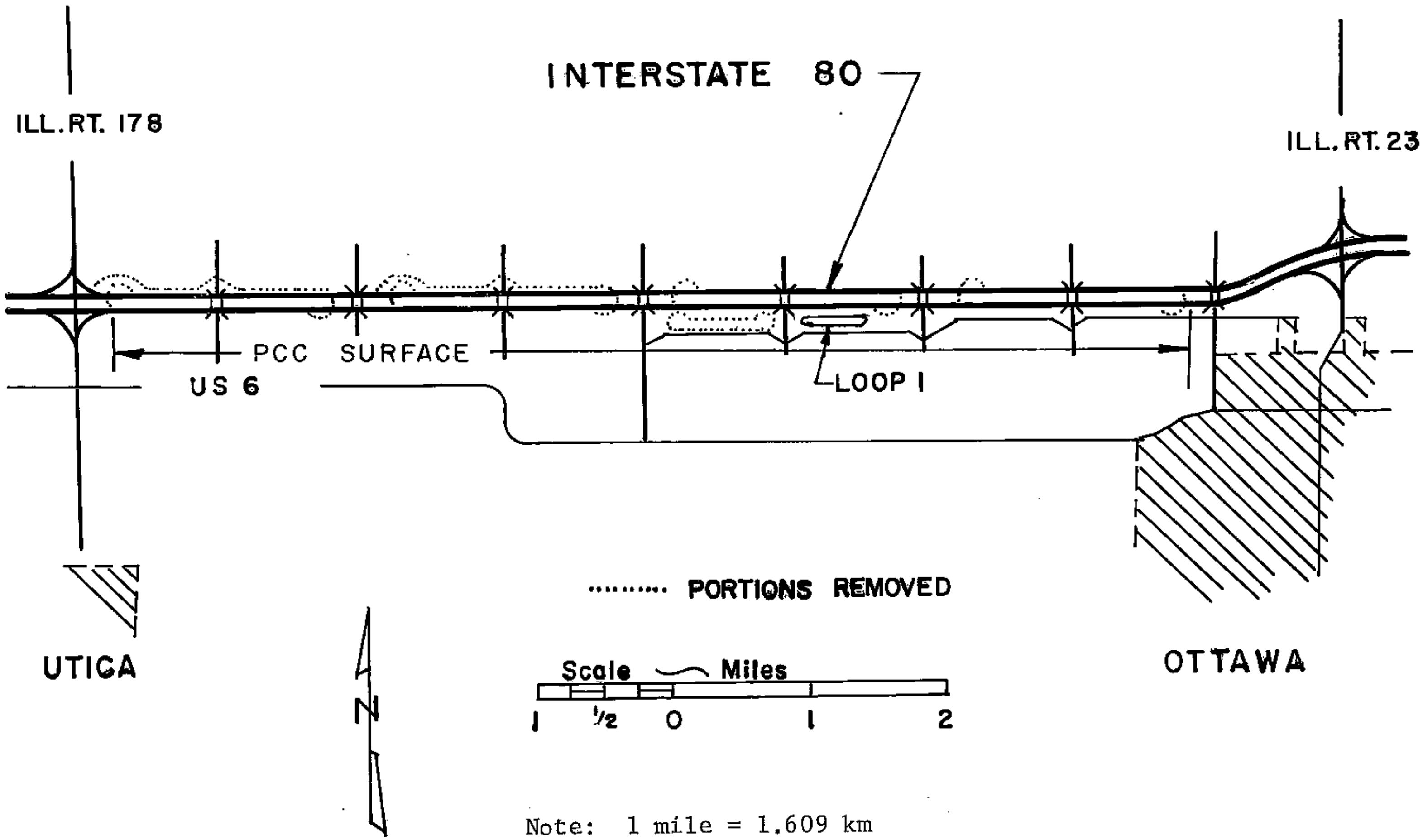


Figure 1. Layout of the Rehabilitated AASHO Test Road.

new flexible sections that represented designs not included in the original test. Gravel (Illinois Grade 7) replaced the AASHO sand-gravel material as subbase in most of the new designs, and crushed stone (Illinois Grade 8) was used as subbase in a few rigid sections. The crushed stone (Illinois Grade 8) was used as base course in one new flexible section. BAM and CAM were used as base in the new flexible pavement designs and as subbase in new rigid pavement designs.

Since the opening of Interstate 80 to regular highway traffic in 1962, the data needed to document the behavior of the experimental pavement sections in the rehabilitated test road have been collected and analyzed.

The behavior of the surviving AASHO Road Test sections under regular mixed highway traffic has been analyzed using the AASHO Road Test equation for describing pavement performance, and by illustrating the value of the equation in describing and evaluating the behavior of the new pavement designs, pavement structural elements and new materials. The results of these analyses are presented herein as the last iterim report of Phase 1, and are intended as final fulfillment of the objectives of the study.

BACKGROUND

The AASHO Committee on Highway Transport (1) recommended that as many as possible of the AASHO Test Road Sections be saved and included in the new Interstate 80 which replaced the original test facility. For inclusion in the rehabilitated roadway, the test sections had to be at least 8.0 in. (203 mm) thick and structurally sound, with no visible signs of deterioration. In the rigid pavement tangents, a few serviceable 8.0-in. sections, most of the 9.5-in. and all of the 11.0-in. and 12.5-in. sections, were retained. In the flexible tangents, test sections that survived the original testing with at least a PSI of 1.5 were resurfaced and included in the study. All others were replaced.

New pavement sections were placed in the test tangents at locations where AASHO sections had been removed and between the tangents to connect them into a continuous roadway through the test facility. The 3-ft embankment was retained intact in the original tangents except where extra depth was required to accommodate new test sections. New embankment to connect the test tangents was also 3 ft (0.9 m) thick. The A-6 embankment soil from the loop turnarounds served as the source of soil material for the new embankments. The granular material from test sections that had to be removed was salvaged, stockpiled, and reused in the new construction. All new materials and construction procedures that were used were required to meet Illinois Standard Specifications for Road and Bridge Construction, adopted in 1958, and the Special Provisions applicable to the Test Road rehabilitation project.

The rehabilitated test facility includes 47 original rigid test sections from the AASHO Test Road and 37 new rigid sections in the eastbound roadway, for a total of 84 rigid experimental sections. The westbound flexible pavement includes 21 original test sections that were resurfaced, together with 22 newer test sections that were built in 1962. Because vehicles normally use the outside lanes, only the experimental sections in the outside lanes have been evaluated.

EXPERIMENTAL DETAILS

This section describes the experimental details and features that bear on the behavior and the performance of the rehabilitated AASHO Test Road.

Test Section Layout

The experimental pavements that compose the present roadway contain all the original rigid sections that were adequate for interstate highway traffic, together with resurfaced original flexible sections and new rigid and flexible sections that represent new designs.

Some new test sections duplicated original Test Road designs even to utilizing salvaged Road Test materials. Other new designs utilized salvaged Road Test materials stabilized with cement (CAM) or asphalt (BAM). A number of new designs utilized Illinois crushed stone (grade 8) and gravel (grade 7) and a few new sections were built with a plain portland cement concrete base and a bituminous surface.

The section numbers of all the portland cement concrete pavement sections are listed in Table 1. In the table, the section numbers are arranged in rows according to subbase type and subbase thickness and in columns according to the thickness of surfacing and use or non-use of pavement reinforcement. The reinforced pavement sections have transverse contraction joints spaced at 40 ft (12.2 m) and 100 ft (30.5 m). All the nonreinforced pavement sections are original AASHO test sections, and they have transverse contraction joints at 15 ft (4.6 m).

All transverse contraction joints were equipped with dowelled load-transfer devices. A number of 8-in. (203-mm) and 9.5-in. (241-mm) reinforced pavement sections and all the 10-in. (254-mm) pavement sections were added to the experimentation in 1962, but all the 11.0-in. (279-mm) and 12.5-in. (318-mm) pavement sections were part of the original AASHO test road. Gravel and crushed stone were used as subbases in the new rigid sections.

Flexible pavement test sections are listed in Tables 2, 3, and 4. Table 2 contains all the original flexible test sections that have been resurfaced.

TABLE 1

TEST SECTION NUMBERS FOR RIGID PAVEMENT DESIGNS

Subbase Type	Subbase Thickness (in.)	PCC Pavement Thickness (in.) Reinforced 8	Reinforced 9.5	Reinforced 10	Reinforced 11	Reinforced 12.5	Non-Reinforced 8	Non-Reinforced 9.5	Non-Reinforced 11	Non-Reinforced 12.5
BAM	4	070,082	072,080	058						5
CAM	4	084,094	086,096	060.						5
SGM	3	692	382,554 646		392,516	360	672	352,512 542,676	364,378 530	396
	4			386						
	5.5			506,550						
	6	670	404,504 544,666	D4C	338,346 546	356	658	368,390 528,702	388,398 498	350
	7			348,508						
	8.5			340						
	9	696	500,668		344,496	358	652	376,690	366.510	380
New SGM 1/	6	092	078							
Gravel	6	068,074	066,076	A2D,A4D 342,384 D2C,490 538,548 520, C2B						
Crushed Stone	6	064,088	062,090							
No Subbase	0							552		

1/ New sand-gravel material that was obtained from an unused stockpile in a pit at the west end of the construction.

Note: 1 in. = 25.4 mm

TABLE 2

TEST SECTION NUMBERS FOR RESURFACED FLEXIBLE PAVEMENT DESIGNS

Surface Thickness Total 1/ (in.)	Surface Thickness Original (in.)	Base Thickness (in.)	SGM Subbase Thickness 8 in.	SGM Subbase Thickness 12 in.	SGM Subbase Thickness 16 in.
9	4	6	578	--	--
9.5	4	6	--	626	--
10	5	6	592	--	--
	4	9	--	478	310
10.5	5	9	--	--	266
11	5,6 2/	3	--	480	256
	5	6	--	582	--
	5,6	9	--	428	334
11.5	6	6	--	258	--
	6	9	272	--	--
12.0	5,6	6	470	--	302
	6,6	9	264	312	--

1/ The total surface thickness comprises the original surface and the new overlay thickness.

2/ Double numbers indicate the original thickness of each test section in the category in consecutive order.

Note: 1 in. = 25.4 mm

TABLE 3

TEST SECTION NUMBERS FOR NEW FLEXIBLE PAVEMENT DESIGNS

Base Course 1/		Subbase Course 1/		Thickness of Bituminous Surface (in.)			
Type	Thickness (in.)	Type	Thickness (in.)	3.0	4.0	4.5	5.0
BAM	8	Gravel	4	-	-	026,020	-
	10	Gravel	4	-	-	042,030,022	-
	12	Gravel	4	-	-	024	-
CAM	10	Gravel	4	-	-	008,006	-
	12	Gravel	4	-	-	040,028,010,004	-
	14	Gravel	4	-	-	002	-
PCC	8	Mixed	Variable	048,034,012	-		-
	9.5	Mixed	Variable	046,036,014			
BC (Salvaged)	6	SGM	12	-	018	-	016
Crushed Stone	8.5	Gravel	23	-	-	038	-

1/ BAM - Bituminous Stabilized Base
PCC - Portland Cement Concrete Base
SGM - Sand-Gravel Material
CAM - Cement Stabilized Base
BC - Crushed Stone Base, Special

Note: 1 in. = 25.4 mm

TABLE 4

TEST SECTION NUMBERS FOR RESURFACED FLEXIBLE PAVEMENTS WITH WEDGE BASES

Surface Thickness Total (in.)	Surface Thickness Original (in.)	Bituminous Stabilized Base Thickness (in.)	SGM Subbase Thickness (in.)	Test Section No.
8.5	3	5.5 - 16	4	460
9.0	3	5.0 - 16	4	464
9.5	4	7.0 - 18	4	284,286

Note: 1 in. = 25.4 mm

In the table, the test sections are arranged by total surfacing thickness, original surfacing thickness, base thickness (Road Test crushed stone), and subbase thickness (Road Test sand-gravel). The flexible test sections that were added to the experimentation in 1962 are listed in Table 3. In Table 3, the section numbers are arranged in order of type and thickness of base course, type and thickness of subbase and thickness of bituminous surfacing. The flexible pavement experiment includes BAM, CAM and plain PCC bases, salvaged Road Test crushed stone and Illinois crushed stone (grade 8) bases, and Illinois gravel (grade 7) subbases. The test sections with wedge-shape bases listed in Table 4 were part of the original AASHO experiment. In these test sections the base course is a bituminous stabilized aggregate mixture that reduces in thickness in the direction of traffic. The rigid test sections in Loop 1 are listed in Table 5 and the flexible test sections in Loop 1 are listed in Table 6.

Each test section in the rehabilitated test road is identified by number on the map in Appendix A which accompanies this report, and both plan and elevation views of each test section are included on the map.

Environmental Conditions

The AASHO Road Test site near Ottawa, Illinois, has a temperate climate. The mean summer air temperature is 76°F (24°C) and the mean winter air temperature is 27°F (-3°C). Although light freezing occurs earlier, frost does not appear in the soil beneath the pavements until late December and disappears again by early March. The average depth of frost penetration is approximately 28 in. (710 mm). The pavements and soil surfaces freeze and thaw a number of times during the winter, and occasionally even the subsoils thaw for short periods during mid-winter.

TABLE 5

TEST SECTION NUMBERS FOR LOOP 1 RIGID PAVEMENTS

Surface Thickness (in.)	Reinforcement 1/	Subbase Thickness 2/ (in.) 0	6
2.5	R	895,897	899,931
	NR	935	933
5.0	R	905	927
	NR	889,893 903,923	891,901 925,929
9.5	R	907,921	887,915
	NR	919	917
12.5	R	883	911
	NR	881,885	909,913

1/ R = Reinforced; NR = Non-reinforced

2/ Sand-Gravel Materials

Note: 1 in. = 25.4 mm

TABLE 6

TEST SECTION NUMBERS FOR LOOP 1 FLEXIBLE PAVEMENTS

Surface Thickness (in.)	1/ Base Thickness (in.)	2/ Subbase Thickness (in.)		
		0	8	16
1	0	857	867	833,841
	6	827	847	839
3	0	859,861 863	829,831 853,869	817 837
	6	825,851 855	819,845 875	821,835 843
5	0	823	865	877
	6	871	849	873,879

1/ Crushed Stone, Special

2/ Sand-Gravel Materials

Note: 1 in. = 25.4 mm

The mean annual precipitation is 34 in. (864 mm) which includes 25 in. (635 mm) of snow. Approximately 65 percent of the precipitation falls during thundershowers in the months from April to September. About 15 percent of the total precipitation occurs as rain, sleet or snow during December, January, and February. Icy driving conditions may exist on an average of 8 days during the winter.

The average monthly maximum and minimum air temperatures and the total monthly precipitation for the test period are charted in Figure 2. Weather during the rehabilitated test period was very similar to weather during the AASHO test period which was reported earlier (3).

The land surface traversed by the test road is a gently undulating till plain with low relief--605 ft (184 m) to 635 ft (194 m) above mean sea level. It is drained by several small streams that empty into the Illinois River. When the most recent Pleistocene glacier melted, the area was inundated by glacial meltwater, which formed a shallow glacial lake (Lake Ottawa) at about 640 ft (195 m) elevation. As a result of the inundation, the till plain to the north of the Illinois River valley contains, in depressions, 3 ft (0.9 m) to 4 ft (1.2 m) of soil and non-calcareous loess-like silt overlying irregularly occurring deposits of yellow, gray, and brown silt with thin beds of sand and clay. A few inches of gravel sometimes border broad shallow till plain waterways, and some local sand and gravel deposits provided materials for construction of the roadway.

The embankment soil is an A-6 "C" horizon material, which contains small amounts of sand as well as a few pebbles and some small boulders. It is representative of the glacial till in the area. As an average, 96 percent of

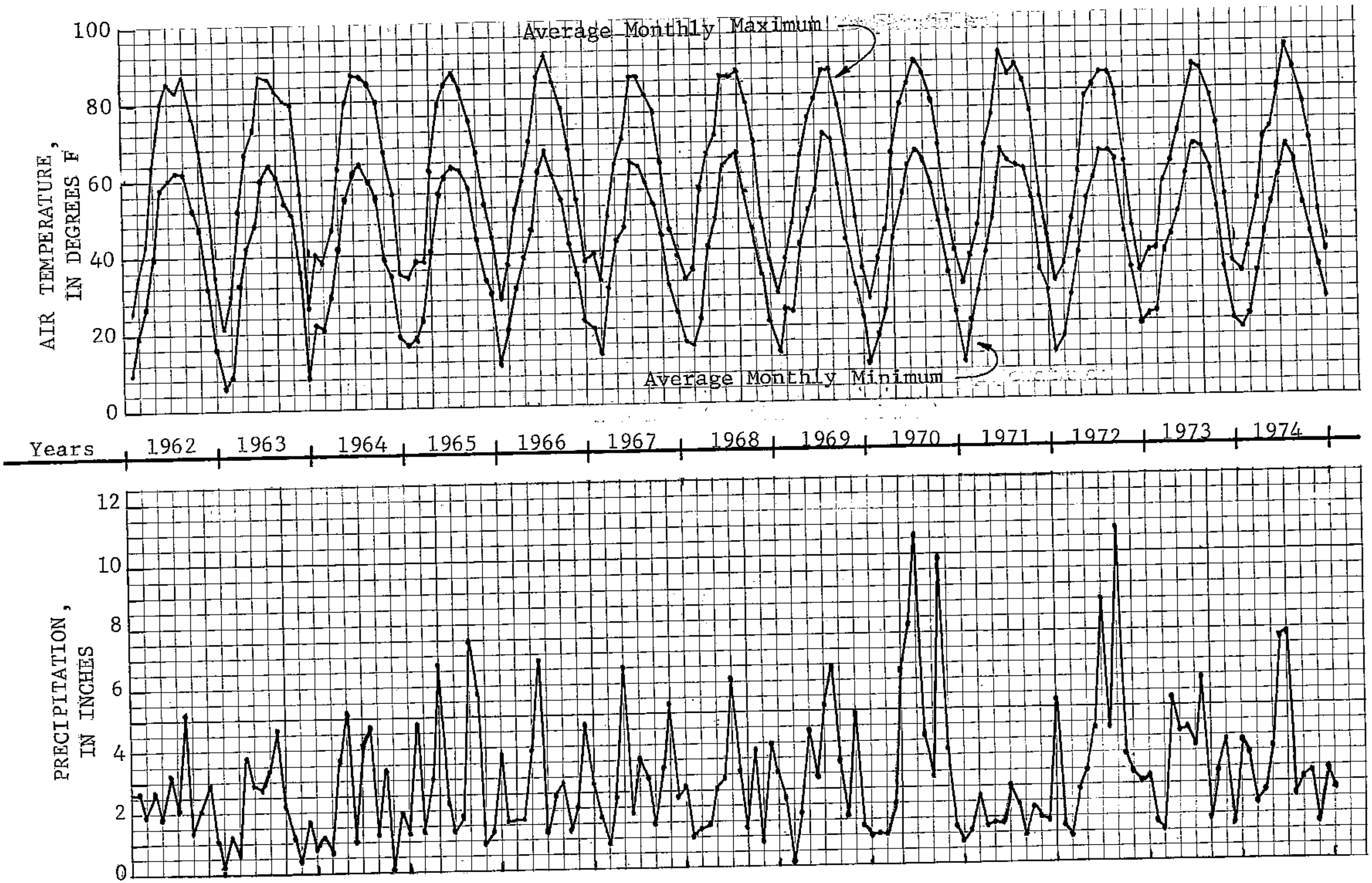


Figure 2. Average monthly maximum and minimum air temperatures and total monthly precipitation at the rehabilitated test road.

the material passed the No. 4 sieve, 75 percent or more passed the No. 200 sieve, and 40 percent of the material was finer than 0.005 mm. The fine clay (less than 0.002 mm) was composed of about 60 percent illite, 30 percent chlorite and 5-10 percent montmorillonite. The mean optimum moisture content was 13.5 percent, and the dry density was 119.2 pcf (1909 kg/m^3) (AASHO T-99). In 1958 the field dry density was 112.5 pcf (1802 kg/m^3), and the field moisture content was 16.3 percent in test block 2 of Loop 1, but by 1973 this same test block had a dry density of 106.5 pcf (1706 kg/m^3) at a moisture content of 14.4 percent at the embankment surface. At 18 in. below the surface, the dry density was 99.4 pcf (1592 kg/m^3) and the field moisture content was 17.3 percent. Traffic has never been permitted to use the Loop 1 pavement sections.

A detailed description of the engineering characteristics for the soils and the materials used in construction of the original test road is contained in HRB Special Report No. 61B (5). The soils and materials that were used in rehabilitation of the test road as Interstate 80 are described in Bureau of Materials and Physical Research Report No. 51 (1). The results of tests on the soils and materials used in the original test road that were included in the International Cooperative Materials Testing Program are contained in HRB Special Report No. 66 (4).

Traffic

The experimental roadway is located entirely between two adjacent interchanges. Therefore, each vehicle that enters the roadway at one interchange must traverse the entire length of the experimental facility in either direction. Although eastbound and westbound traffic may vary daily, on a yearly basis they are almost equal. Hence, the regular highway traffic which has travelled over the experimental road since 1962 is assumed the same for each test section in the experimental roadway.

Since November 1962 when the rehabilitated test road was opened to regular highway traffic, over 25.1 million vehicles have used the roadway. In addition to the regular highway traffic, each original test section received 1.114 million axle loads of a specified type and weight during the AASHO Road Test.

The average daily traffic (ADT), approximately equal in either direction, amounted to 3500 vehicles per day in 1962 when the roadway was opened to regular traffic. Since 1962, the ADT has increased steadily, and by 1973 it had reached 15,700. Due to the fuel shortage, which developed in 1974, the ADT dropped to 14,000, but then began to increase again as the fuel shortage eased. The average daily traffic for the period from 1962 to 1974 is shown in Figure 3.

The traffic stream consists of 71 percent passenger cars, 6 percent single-unit trucks and 23 percent multiple-unit trucks. The composition has remained fairly constant over the years, although daily, weekly, and seasonal variations occur. For example, the number of passenger cars using the roadway fluctuates with daily movements of commuters to and from places of employment, holiday movements, vacation travel, etc. On the other hand, the number of trucks using the roadway is relatively constant except for weekend periods when the number of trucks declines.

Normally, vehicles use the outside traffic lane. However, during peak travel periods up to 25 percent of the passenger cars and 4 percent of the trucks shift to the inner passing lane. More than 96 percent of the trucks, which generate the heavy axle loads, use the outer traffic lane continually.

Since the original test sections have carried both AASHO Road Test traffic, which was standardized in type and magnitude, and regular highway traffic which

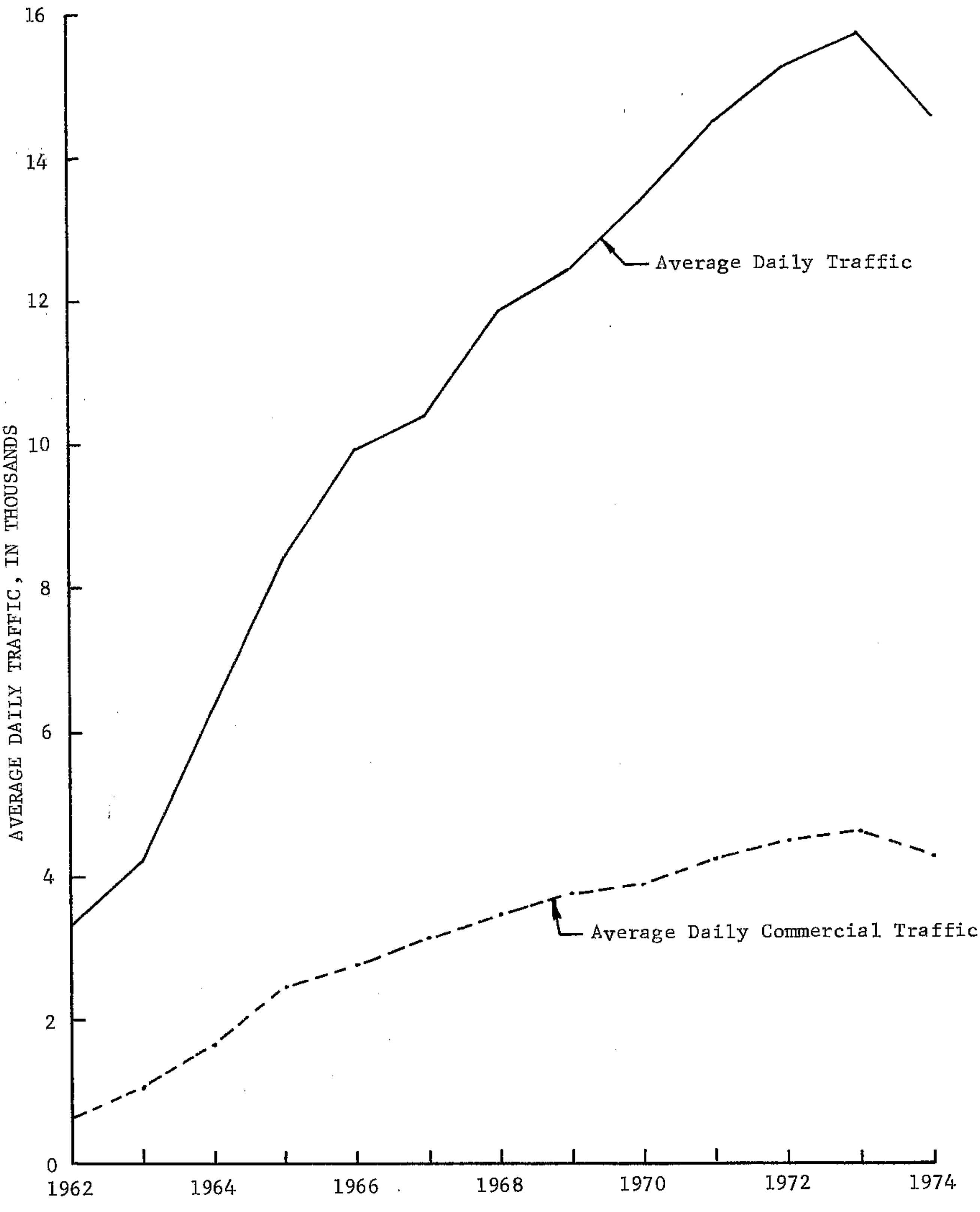


Figure 3. Average Daily Traffic and Average Daily Commercial Traffic from 1962 to 1974.

was the same for all test sections, it was necessary to convert both types of traffic to a common basis that would permit comparison. The 18-kip single-axle load equivalency factors developed from the Road Test Performance equations (5) were used for this purpose.

Data from traffic counts and load studies on I-80 at the Road Test site provided the basic information for establishing the number of equivalent 18-kip single-axle load applications. Composition of the traffic stream was determined by visual counts made over one 24-hour period each month from November 1962 through October 1970. The day of the week on which the counts were made varied from Monday through Friday. A few counts also were made on Saturday and Sunday to determine the composition of weekend traffic. The number of passenger cars and trucks was recorded by travel direction and by lane usage (traffic and passing). Trucks were sorted into one of ten appropriate vehicle classes as recommended by the Bureau of Public Roads, now the Federal Highway Administration. In May 1965, a magnetic loop detector which provided continuous count data in both directions was installed. After the installation of the detector, the visual count data were used to monitor the composition of the traffic stream and to distribute the continuous count data into vehicle classes.

Load studies also were conducted periodically in cooperation with the local DOT district office. A load study was conducted by channelizing the traffic stream to sort out the trucks, which were then directed at creep speed (3 mph, 5 km/hr) over a set of electronic platform scales that had been permanently installed in the pavement. Truck type and axle weights were also recorded. From these data an adjustment factor was derived that was used to convert the traffic count data into an equivalent number of 18-kip single-axle load applications. The total number of 18-kip equivalent single-axle load applications that were applied to the new rigid and flexible test sections, and to the original rigid AASHO test sections up to 1974, is listed in Table 7.

Measurement Program

The measurement program that began in 1962 provided data for evaluating the performance of the surviving AASHO test sections under mixed highway traffic, and provided a basis for evaluating the behavior of the new pavement sections and the new materials that had been added to the experimentation. Surveys of pavement smoothness, cracking, patching, and rut depth (flexible pavements only) were conducted annually. The measurement techniques that were used were identical to those that had been employed during the AASHO Road Test.

As an adjunct to the pavement performance studies, slab faulting, winter joint openings, pavement blowups, pumping, and spalling were measured and recorded. Static rebound deflections by Benkelman beam at edges and at corners of rigid pavement slabs and in the wheelpaths of flexible pavements also were measured annually until 1970. Coring studies were conducted to determine the condition of the stabilized subbases under the pavement joints and edges, and two pavement sections containing typical joints were removed from rigid pavement sections to determine the condition of the load-transfer dowels.

Pavement smoothness was measured with the Illinois Roadometer (2, 3). The roadometer output is called a Roughness Index (RI) and represents inches of measured roughness per mile of pavement. A correlation of the Illinois Roadometer with the AASHO Road Test Profilometer (2) established a relationship between the two instruments that made it possible to compute Present Serviceability Index (PSI) values for each test section in the rehabilitated test road.

TABLE 7

CUMULATIVE 18-KIP EQUIVALENT SINGLE-AXLE LOAD APPLICATIONS 1/
AT THE REHABILITATED AASHO TEST ROAD

	New Test Sections		Original Rigid Test Sections		
Year	Rigid	Flexible	Loop 4	Loop 5	Loop 6
1959 2/	-	-	569,756	1,424,058	3,006,972
1960	-	-	1,640,571	4,090,847	8,642,793
1963 3/	222,595	164,920	1,863,166	4,313,432	8,865,388
1964	601,648	438,082	2,242,219	4,692,495	9,244,441
1965	1,196,817	844,782	2,837,388	5,287,664	9,839,610
1966	1,829,781	1,274,619	3,470,352	5,920,628	10,472.574
1967	2,904,309	2,003,387	4,544,880	6,995,156	11,547,102
1968	3,854,544	2,634,317	5,495,115	7,945,391	12,497,337
1969	4,885,412	3,289,713	6,525,983	8,976,259	13,528,205
1970	5,873,824	3,948,152	7,514,395	9,964,671	14,516,617
1971	6,906,750	4,648,180	8,547,321	10,997,597	15,549,543
1972	8,001,432	5,389,534	9,642,003	12,092,279	16,644,225
1973	9,124,499	6,130,890	10,765,070	13,215,346	17,767,292
1974	10,000,852	6,724,385	11,641,421	14,091,699	18,643,645

1/ Based on Equivalency factors for p = 2.5 (2)
2/ Includes traffic applied during 1958
3/ Includes traffic applied during 1962

The pavement condition surveys identified a number of types of pavement distress. The alligator-type cracking in bituminous pavements and the transverse and longitudinal cracking in rigid pavements, as well as patching, were identified and recorded using the AASHO Road Test procedures (3). Transverse and longitudinal cracking, which occurred in the bituminous pavements of the rehabilitated road that overlaid stabilized and plain portland cement concrete bases, were recorded in the same manner as those in the rigid pavements. In addition, D-cracking, spalling, ravelling, compression cracking, corner breaks, blowups, and pumping in the rigid pavements, and belt cracking, spalling and ravelling in the bituminous pavements, were identified in accordance with Illinois Division of Highways procedures.

For the pavement performance studies, the Class 3 and Class 4 transverse and longitudinal cracking in the rigid pavements were recorded as lineal feet of cracking per 1000 sq ft of pavement. The Class 2 and Class 3 alligator-type cracking in bituminous pavements and the bituminous patches were recorded as square feet of cracking and of patching per 1000 sq ft of pavement.

The rehabilitated AASHO Test Road represents a fairly extensive adjunct to the original AASHO experiment. An insight into the applicability of the data from the measurement program is essential to an understanding of the validity and significance of the findings. Extensive analysis of the data from the main factorial experiment in the original test (3, 4, 5) has shown that (1) certain interactions studied in the pavement performance experiment may be considered negligible, (2) results similar to those that were obtained in the main experiment could be expected from partial rather than full factorial designs in subsequent satellite studies, and (3) only a small fraction of the test section array was needed to test any similar sets of design variables in subsequent studies, particularly in the same climatic environment and on the same foundation soils as was the original experiment. These findings have been taken to mean that in the pavement performance studies only a sufficient number of test sections to determine the means and the trends of the means were needed to establish the desired relationships and to yield a reasonable degree of confidence in the results. There are several important exceptions. First, in the rehabilitation of the test road as an Interstate highway, all the rigid pavement test sections that were less than 8 in. (203 mm) thick and any thicker sections that had been damaged by the test traffic were removed and were replaced with new pavement. Thus, the experimental pavement sections that were retained represented sections of pavement that were above normal in performance in their design groups. Second, none of the original flexible pavement sections were retained without overlays, and only a limited number of designs were replicated. Third, none of the special studies represented a full factorial design. Therefore, no tests for interactions of the variables in the special studies could be carried out.

PAVEMENT PERFORMANCE

At the AASHO Road Test, pavement performance was defined as the serviceability trend of a pavement in relation to increasing numbers of axle load applications (4). The present serviceability of a pavement was determined by combining, mathematically, certain physical measurements from a large number of pavements in such a way as to give the Present Serviceability Index (PSI) which represented the mean of a great many individual Present Serviceability

Ratings (PSR) of the pavements by experienced highway users. The serviceability trend of the pavements was a continuous graph of the Present Serviceability Index determinations in relation to axle load applications.

The PSI was obtained as the mathematical solution of two separate equations: one applies to rigid (portland cement concrete pavements) and the other applies to flexible (bituminous concrete pavements). The equations are as follows:

(a) For rigid pavements:

$$PSI = 5.41 - 1.8 \text{ Log } (1 + \overline{SV}) - 0.09 \sqrt{C + P}$$

(b) For flexible pavements:

$$PSI = 5.03 - 1.91 \text{ Log } (1 + \overline{SV}) - 0.01 \sqrt{C + P} - 1.38 \overline{RD}^2$$

In these equations:

PSI = Present Serviceability Index

$\overline{SV}$ = the mean slope variance in longitudinal profile as measured by the AASHO Profilometer.

C = the total lineal ft of crack projection in 1000 sq ft of rigid pavement or the total area in sq ft of grid-type cracking in 1000 sq ft of bituminous pavement. (4)

P = the total area in sq ft of any type of patching in 1000 sq ft of pavement.

$\overline{RD}^2$ = The square of the mean rut depth in two wheelpaths.

Widespread use of the pavement performance-serviceability concept was impractical until a reliable and speedy method for measuring slope variance could be found that would replace the AASHO profilometer. The BPR-type roadometer proved to be Illinois' solution to the problem. The Illinois Roadometer (2) measured upward vertical movement of a rubber-tired wheel in relation to the trailer frame of a single-wheel trailer, which was towed at

20 mph (32 km/hr). The output is called Roughness Index (RI). The correlation study (2), which related the Roadometer output to that of the AASHO Profilometer, gave the following equations for Illinois pavements:

For rigid pavements:

$$PSI = 12.0 - 4.27 \, Log \, \overline{RI} - 0.09 \sqrt{C + P}$$

For flexible pavements:

$$PSI = 10.91 - 3.90 \, Log \, \overline{RI} + 0.01 \sqrt{C + P} - 1.38 \, \overline{RD}^2$$

where:

PSI = Present Serviceability Index

$\overline{RI}$ = Mean Roughness Index

and all other terms are as previously defined.

By substituting appropriate values for the terms in the equations, the Present Serviceability Index of both rigid and flexible pavements can be determined at any point in their service life. Then, by relating the Present Serviceability Index to the corresponding number of axle load applications, a curve, which passes through the Present Serviceability Index points, can be drawn to describe the serviceability trend or performance of the pavements.

Furthermore, the curves can be terminated at some pre-selected terminal PSI level to represent the end of the design service life. At the AASHO Road Test a terminal PSI of 1.5 was selected. For interstate highways in Illinois, a PSI of 2.5 is considered terminal and for other Illinois primary highways, a PSI of 2.0 is considered terminal.

In this section of the report, the pavement serviceability-performance concept has been applied to the original AASHO test sections that were retained as part of the rehabilitated test road and to the new test sections that were added to the experimentation in 1962 to accomplish the following purposes:

first, to document the performance of the original rigid AASHO test sections under mixed highway traffic and thereby test the validity of the performance equations; second, to document and to evaluate the performance of rigid test sections with crushed stone, gravel, and stabilized aggregate subbases that were added to the experimentation in 1962; third, to evaluate the performance of rigid and flexible test sections that were built in 1962 as duplicates of original AASHO test sections; fourth, to evaluate the performance of bituminous overlays on original flexible AASHO test sections; fifth, to evaluate the performance of new flexible designs with new crushed stone and new stabilized aggregate base courses added in 1962; and sixth, to evaluate the performance of the bituminous pavements with plain portland cement concrete bases that were built in 1962.

Rigid Pavement Performance

The experimental rigid pavement sections are located in the eastbound lanes of the test road (Fig. 1 and App. A). In addition to the rigid pavement sections that were added in 1962, there are 47 AASHO test sections, six that have slabs 8.0 in. (203 mm) thick, 20 that have slabs 9.5 in. (241 mm) thick, 15 that have slabs 11.0 in. (279 mm) thick, and six that have slabs 12.5 in. (318 mm) thick. Each of the original AASHO test sections, except section No. 552, has a subbase of the original Road Test sand-gravel mixture which is 3 in., 6 in., and 9 in. (76 mm, 152 mm, and 229 mm) thick. Since it had been shown (4) that subbase thickness had no effect on pavement performance at the AASHO Road Test but that the presence of a subbase improved performance, the test sections have been placed in groups that only differ in surface thickness. The test section without a subbase is not included in the groups.

All of the test sections have received the same number of equivalent 18-kip single-axle load applications since 1962. During the AASHO Road Test, single-axle trucks used the inside lanes and tandem-axle trucks used the outside lanes of each test loop. The loads varied between lanes and among the test loops (4). Only the 9.5-in. (241-mm) surface thickness was repeated in all three of the original test loops (4, 5, and 6). Although each 9.5-in. section received the same number of load applications (1.114 million), the equivalent number of 18-kip single-axle load applications differed depending on the loading that was applied to each test loop (Table 7).

The observed performance of the rigid AASHO test sections is compared to the performance predicted for them by the Road Test equation in Figures 4, 5, 6, and 7. Test sections that represented duplicate Road Test designs were built 8.0 in. (203 mm) and 9.5 in. (241 mm) thick. They are included also in Figures 4 and 5. The comparison was made by computing the PSI for each test section using pavement smoothness, cracking and patching data from pavement surveys and relating to the corresponding total 18-kip ESAL applications ($\mathrm{Log}W_{18}$) for the years 1962, 1972, and 1974. The performance curves were obtained by using the AASHO performance equation to compute the ($\mathrm{Log}\ W_{18}$) values for an appropriate number of PSI levels.

The performance of the six original AASHO 8.0-in. pavement sections and one duplicate section added in 1962 is shown graphically in Figure 4. As can be seen in the Figure, the 8.0-in. pavement sections all performed better than expected according to the performance curve for 8.0-in. pavement sections because only those 8.0-in. test sections that were in the best condition at the end of the AASHO Road Test were retained for this study. Thus, these six

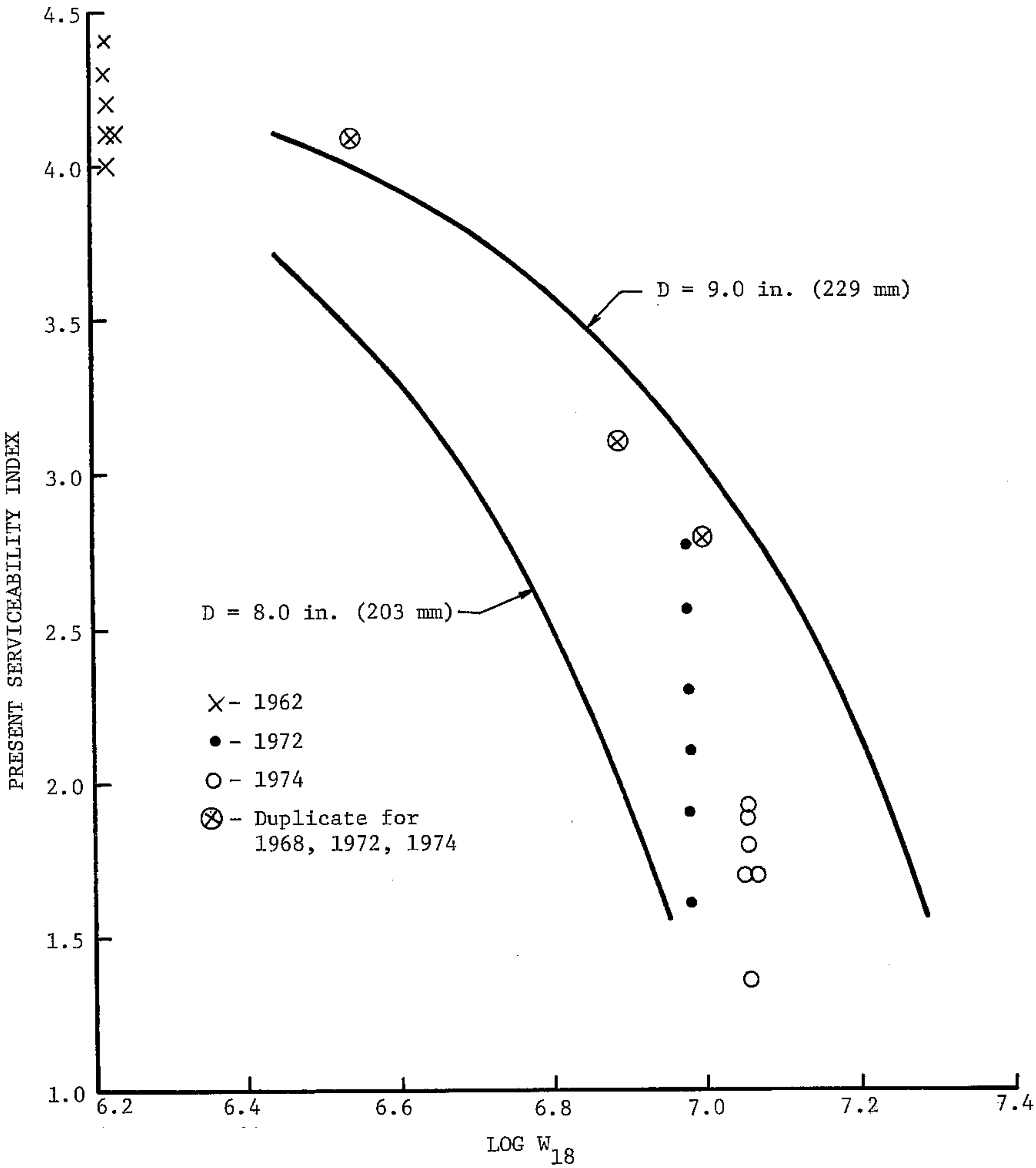


Figure 4. Performance of 8.0-in. rigid AASHO sections.

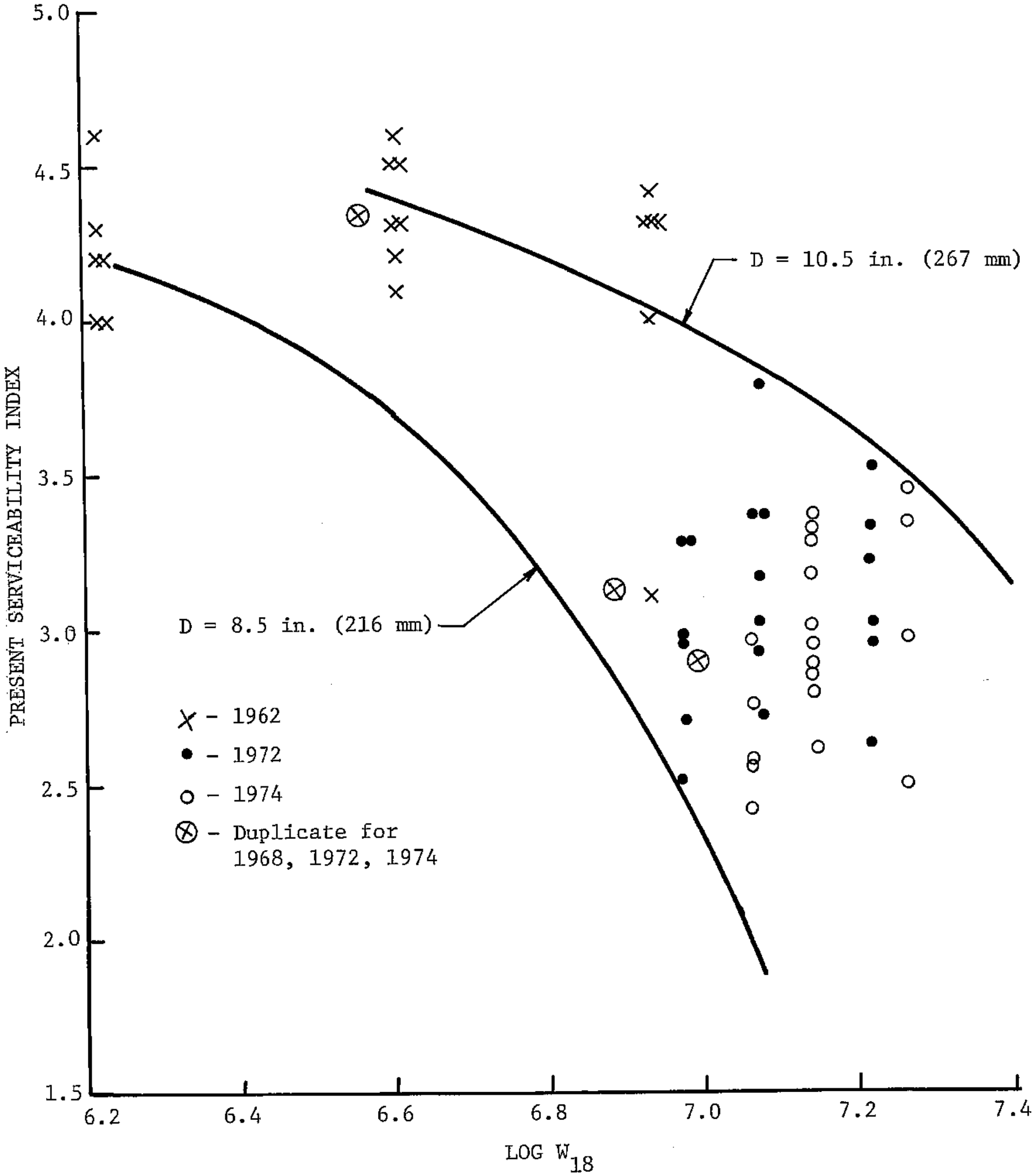


Figure 5. Performance of 9.5-in. rigid AASHO sections.

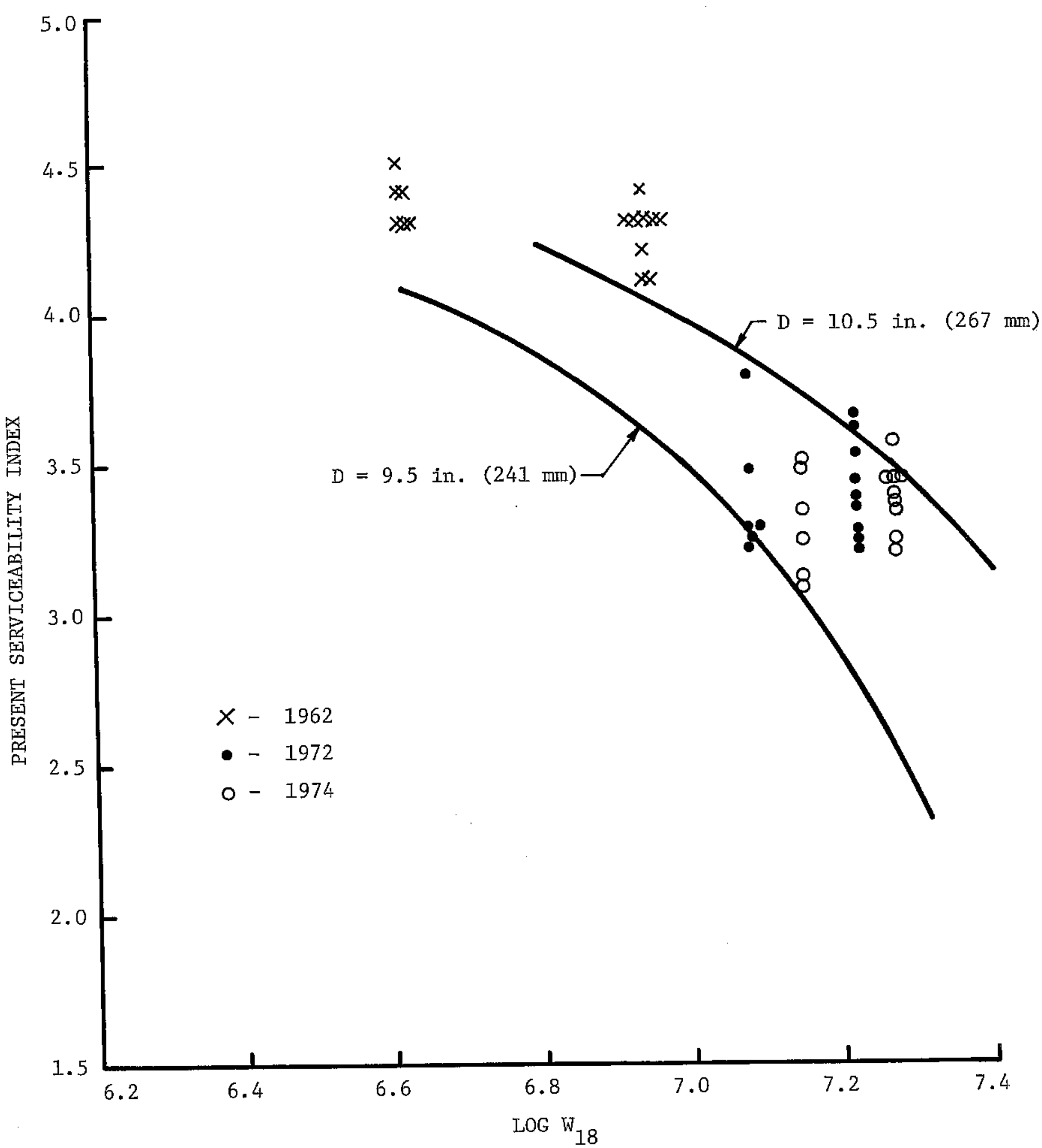


Figure 6. Performance of 11.0-in. rigid AASHO sections.

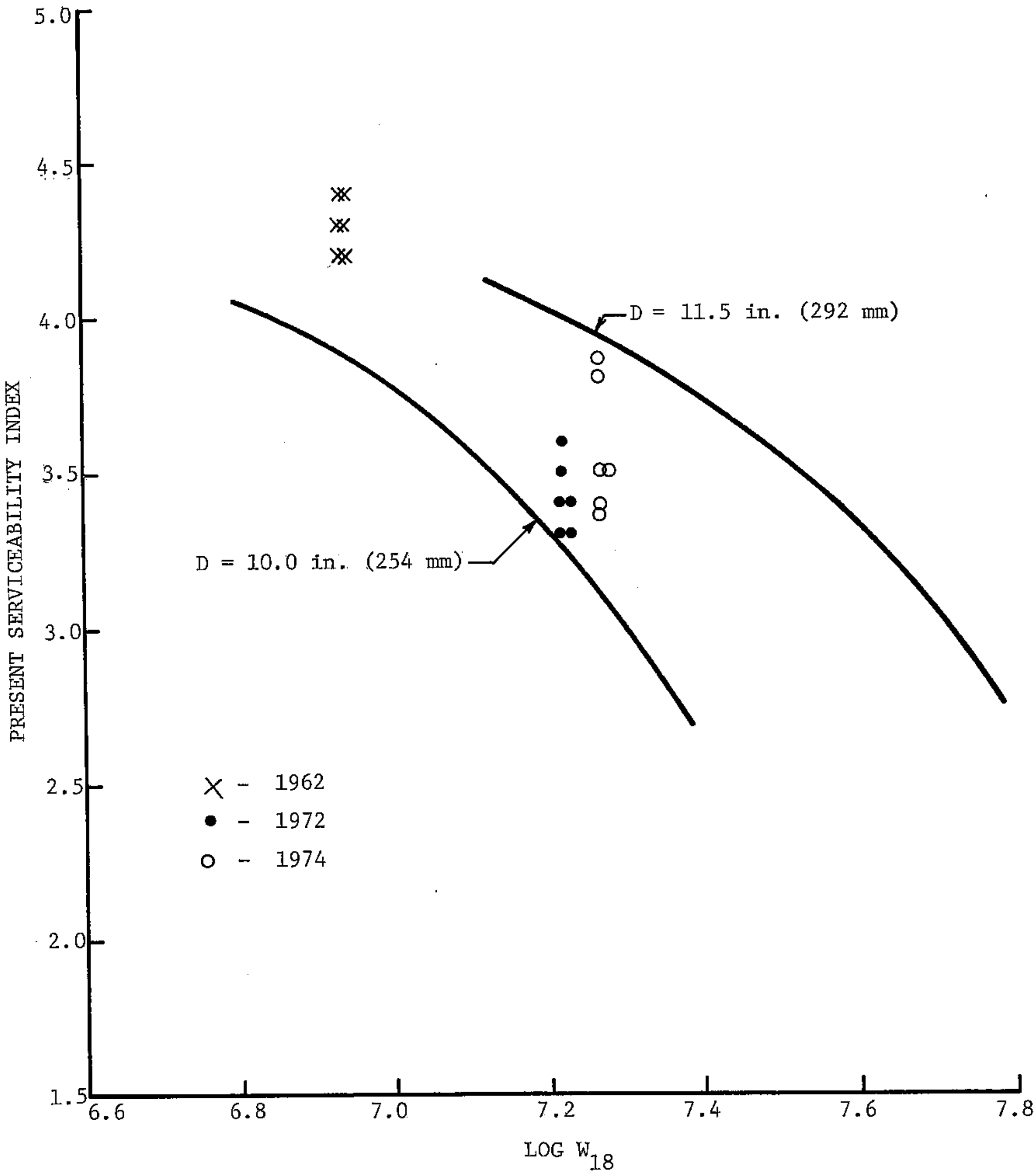


Figure 7. Performance of 12.5-in. rigid AASHO sections.

sections were among the best performing sections that yielded data for derivation of the Road Test performance equation. It also can be seen in Figure 4 that the level of serviceability continued to be better than normal throughout the service life of the 8.0-in. sections. Analysis of the residual variation in LogW at the Road Test (Ref. 4, pages 154, 155) showed that a variation of $\pm$ 0.34 LogW included 90 percent of the PSI observations. The PSI points plotted in the chart fall within the range. Therefore, it is concluded that even though the group of 8-in. pavement sections were above normal in serviceability during their service life, their behavior was within the range of the Road Test equation. Using a terminal PSI of 2.5, it can be seen also in Figure 4 that four out of the six 8-in. AASHO test sections were below the terminal serviceability level in 1972, and by 1974 all of the sections were below a PSI of 2.0. The 8-in. test sections based on a terminal PSI of 2.5 had reached the end of their service life by 1972 after receiving a total of about 9.6 million 18-kip ESAL applications.

The performance of the 9.5-in. test sections is shown in Figure 5. This group includes 19 original AASHO test sections and one duplicate test section that was added to the experimentation in 1962. As can be seen in the figure, the PSI points are uniformly distributed about the performance curve that was computed from the Road Test equation. This is taken to mean that the group of original 9.5-in. AASHO test sections was a more representative sample than those in the 8.0-in. group and that the Road Test performance equation also gave a good estimate of the performance expectation for the 9.5-in. sections. Moreover, the PSI of the 9.5-in. test sections in 1974 ranged from 2.4 to 3.4. In fact, only two of the sections dropped below a PSI of 2.5. The total 18-kip

ESAL applications at the terminal PSI of 2.5 for the 9.5-in. sections is indicated as approximately 18.6 million.

The performance of the 11.0-in. AASHO test sections is shown graphically in Figure 6. These 15 test sections are all the 11-in. AASHO sections that were in the outside lane of the original experiment. As can be seen in Figure 6, the serviceability trend of the 11.0-in. sections is established, but as of 1974 none of the sections had a PSI below 3.1. The AASHO performance curve that would best fit the data appears to be the curve for the 10.0-in. (254-mm) design thickness.

The performance of the 12.5-in. AASHO test sections is shown graphically in Figure 7. These six test sections were all in the outside lane of former Loop 6 of the original experiment. As can be seen in Figure 7, none of the 12.5-in. sections had a PSI below 3.3 by 1974. The serviceability trend is not as well defined as other pavement thicknesses after 18.6 million 18-kip ESAL'S had been applied. Even though the serviceability trend is not well defined, the 12.5-in. sections were underperforming the expectation of the performance equation. The performance curve that appears to fit the data best is the one for a pavement design that is approximately 10.75 in. (273 mm) thick.

Revised Performance Equation

As can be seen in Figures 4 through 7, the Road Test performance equation failed to predict the serviceability trend for the 11.0-in. and 12.5-in. rigid pavement sections with the same precision that was achieved for the 8.0-in. and 9.5-in. pavements. The difference in precision was due to the indefinite nature of the serviceability trend that had developed in the thicker pavements by the time that the original test was closed. By 1974, when testing under regular highway traffic was discontinued, the 11.0-in. and 12.5-in. pavements had developed a more pronounced serviceability trend, although they had not as yet reached a terminal serviceability level. To improve the precision of the original Road Test equation for the 11.0-in. and 12.5-in. pavements, new data, obtained during the post Road Test research, have been used in conjunction with original data to develop a revised rigid pavement performance equation.

For the revision, the data for the 8.0-in. pavements were supplemented with data from the original test to offset the bias that had been introduced in 1962 by removing all but six of the best performing 8.0-in. pavement sections from the test. Criteria for selection of the supplemental 8.0-in. data were: (1) that only data from test sections from the outer traffic lane be used; and (2) that a substantial serviceability trend had developed in each section during the Road Test. Ten test sections from the Road Test met the criteria and were added to the group of 8.0-in. pavement sections.

In the computation of the original performance equation, at least five sets of coordinates on the serviceability-application history curves were used for each section. For the test sections that had reached a terminal serviceability level of 1.5, the coordinates were for PSI levels of 3.5, 3.0, 2.5, 2.0, and 1.5. Some test sections that had not reached a terminal serviceability level of 1.5 also were used. For these sections, five coordinates from evenly spaced time periods during the test were selected. To be consistent with the original procedure, five time coordinates also were selected in developing the revised performance equation. In most cases these were data points for 1968, 1969, 1971, 1972 and 1974. However, a few sections had a PSI of less than 3.3 in 1968. For these sections, the 1968 data were replaced with data from an earlier date when the PSI was at least 3.5. All data that were used to supplement the 8.0-in. test section group in the derivation of the revised equation were used also in the derivation of the original equation.

A complete performance equation, involving both load and design parameters, could not be developed since the traffic on the rehabilitated pavements was a normal mixture of vehicles and not controlled loadings as it was during the Road Test. The loading relationships developed from the original equations were assumed to be correct, and the accumulated 18-kip single-axle load applications previously discussed were used as the measure of traffic loading. The original Road Test data used to supplement the 8-in. data were similarly converted to equivalent 18-kip single-axle load applications.

The general form of the original Road Test performance equation is:

$$\log W = \log \rho + \frac{G}{\beta}$$

in which W is the number of load applications, ρ and β are complex functions of design and load, and G is the serviceability loss term. For rigid pavements the expressions for ρ, β, and G are:

$$\rho = \frac{10^{5.85}\ (D_2 + 1)^{7.35}\ L_2^{\ 3.28}}{(L_1 + L_2)^{4.62}}$$

$$\beta = 1 + \frac{3.63\ (L_1 + L_2)^{5.20}}{(D_2 + 1)^{8.46}\ L_2^{\ 3.52}}$$

$$G = \log \frac{4.5-P}{3}$$

where L_1 = axle load in kips

L_2 = 1 for single axles and 2 for tandem axles

D_2 = slab thickness in inches

p = Present Serviceability Index

For the 18-kip single-axle load, the log ρ and β terms reduce to:

$$\log \rho = -0.058 + 7.35 \log (D_2 + 1)$$

$$\beta = 1 + \{10^{7.209} \times (D + 1)^{-8.46}\}$$

Within the range of thicknesses involved in this study, the β term for an 18-kip single-axle load ranges from 1.137 for an 8-in. slab to 1.004 for a 12.5-in. slab. As a simplication for this analysis, β was assumed to be equal to 1.0 for all thicknesses. Using W_{18} for the number of 18-kip single-axle applications, this reduces the original performance equation to:

$$\log W_{18} = -0.058 + 7.35 \log (D_2 + 1) + G$$

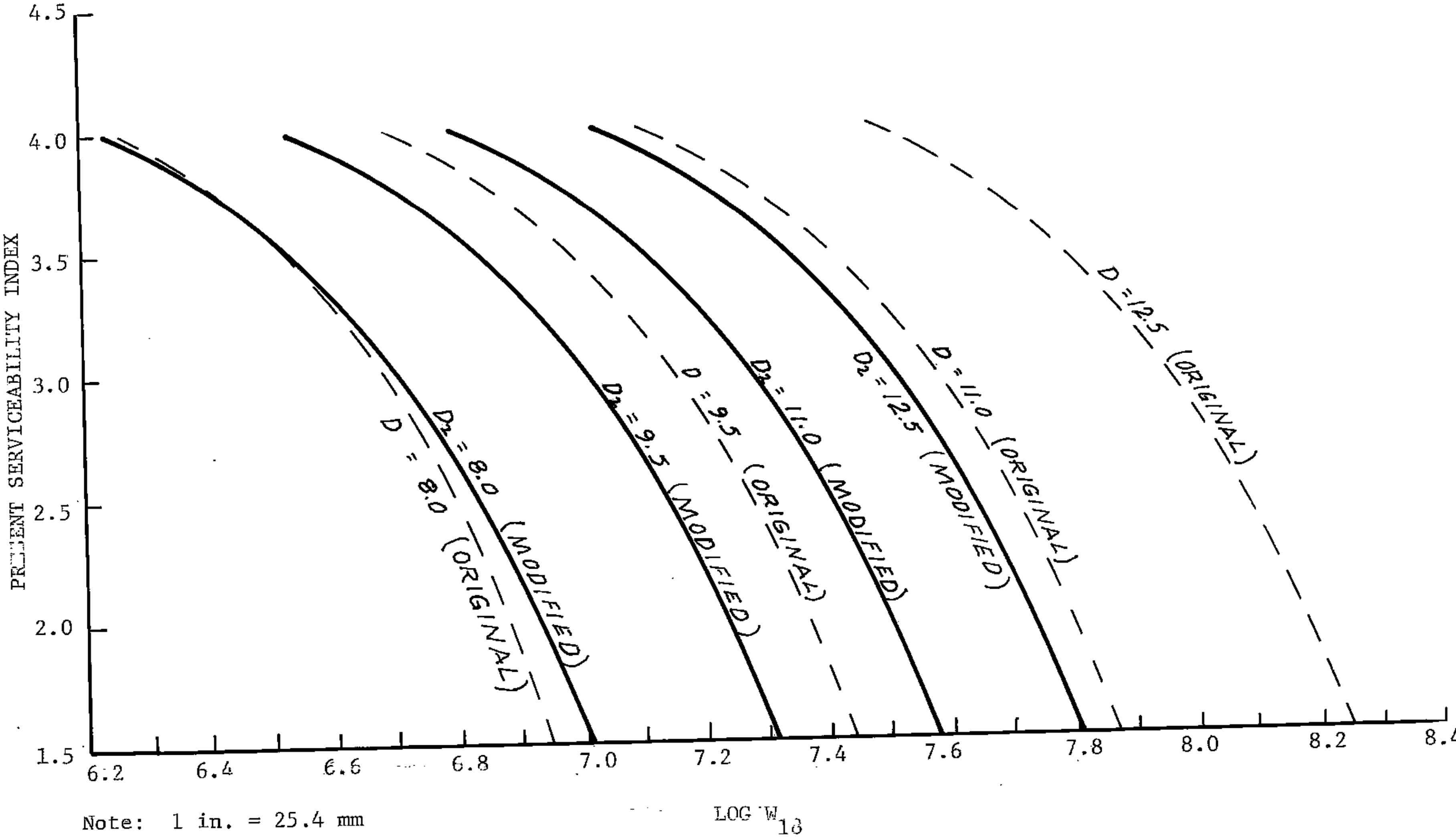
Substituting a and b for the constants, this equation can be written as:

$$(\log W_{18} - G) = a + b \log (D_2 + 1)$$

In this form the data from this study were used to develop revised values for a and b. Using $(\log W_{18} - G)$ as the dependent variable and $\log (D_2 + 1)$ as the independent variable, a correlation and regression analysis was performed. The analysis showed a correlation coefficient of 0.77 and a standard error of estimate for the regression equation of 0.21. Written in a form similar to the original performance equation, the modified equation is:

$$\log W_{18} = 2.724 + 4.50 \log (D_2 + 1) + G$$

Performance curves by the modified equation are plotted in comparison to the curves given by the original equation for each slab thickness in Figure 8. As can be seen in the figure, the original curves predict progressively greater performance for the sections than do the modified curves.



Note: 1 in. = 25.4 mm

Figure 8. Comparison of pavement performance curves by original and modified equations.

For a comparison with the Road Test performance equation, the root mean square residual of $\log W_{18}$ for the revised equation was determined. For all the data it was found to be 0.21. For the individual thicknesses studied, it was found to be 0.23 for the 8-in. sections, 0.22 for the 9.5-in. sections, 0.13 for the 11-in. sections, and 0.11 for the 12.5-in. sections. These compare quite favorably with the root mean square residual of 0.22 for logW reported for the Road Test equation (4), indicating that the revised equation predicts the performance of the test sections retained in the rehabilitation about as well as the original equation predicted the performance of the test sections during the Road Test.

New Subbase Materials

As an adjunct to the rigid pavement research in the rehabilitated AASHO Road Test, four additional kinds of subbase materials were added to the experimentation. The new subbase materials were gravel (Ill. grade 7), crushed stone (Ill. grade 8), salvaged Road Test sand-gravel material stabilized with asphalt (BAM), and salvaged Road Test sand-gravel material stabilized with cement (CAM). The new materials were used as subbase under new rigid pavements that were 8.0 in., 9.5 in., and 10.0 in. (203 mm, 241 mm, and 254 mm) thick. All the new granular subbases were placed 6 in. (152 mm) thick and BAM and CAM subbases were placed 4 in. (102 mm) thick. The materials were prepared and placed in accordance with Illinois Standard Specifications for Road and Bridge Construction and the Special Provisions for the AASHO Test Road Rehabilitation Project. All the pavement sections had sawed, dowelled, contraction joints. The 8.0-in. and 9.5-in. pavements had joints every 40 ft (12.2 m) and the 10-in. pavements had joints every 100 ft (30.5 m). The gradation, preparation, mixing and placement of the materials are described in Illinois Physical Research Report No. 51 (1). In addition to the new subbase materials, the original Road Test sand-gravel subbase material was used as subbase in locations where damaged AASHO sections had been removed and were replaced with 10-in. pavement. In these sections, the existing subbase was cut down in thickness to accommodate the new 10-in. surfacing. One additional new test section with an 8.0-in. pavement and another with a 9.5-in. pavement also were built on 6-in. subbases of the unused sand-gravel material from the original stockpile.

Altogether, the subbase experimentation involved nine new test sections with 8.0-in. pavement, nine with 9.5-in. pavement, and 19 new sections with 10-in. pavement. These included five test sections each on BAM and CAM as subbases, 14 on gravel (grade 7), four on crushed stone (grade 8), seven on the original subbase pavement and two duplicate test sections on the AASHO sand-gravel subbase material from the original stockpile.

By 1974, the new pavement sections had received approximately 10 million applications of 18-kip equivalent single-axle loads as regular mixed highway traffic, but only the 8-in. pavement sections had deteriorated to the point where they were nearing the end of their service life (terminal PSI = 2.5). Two of the 8-in. sections on the granular subbase had a PSI below 2.5. The remaining 8-in. sections had a PSI of 3.1 or higher, and none of the thicker pavement sections had deteriorated below a PSI of 2.7. The performance of the test sections is shown graphically in Figures 9, 10, 11, 12, 13, and 14 arranged by type of subbase material and pavement thickness.

Performance of the 8.0-in. pavement sections is plotted in Figure 9. There are two test sections each on the crushed stone (grade 8) and gravel (grade 7), and another on the AASHO sand-gravel mixture. The data are for the years 1968, 1969, 1971, 1972, and 1974. The plotted PSI points show the

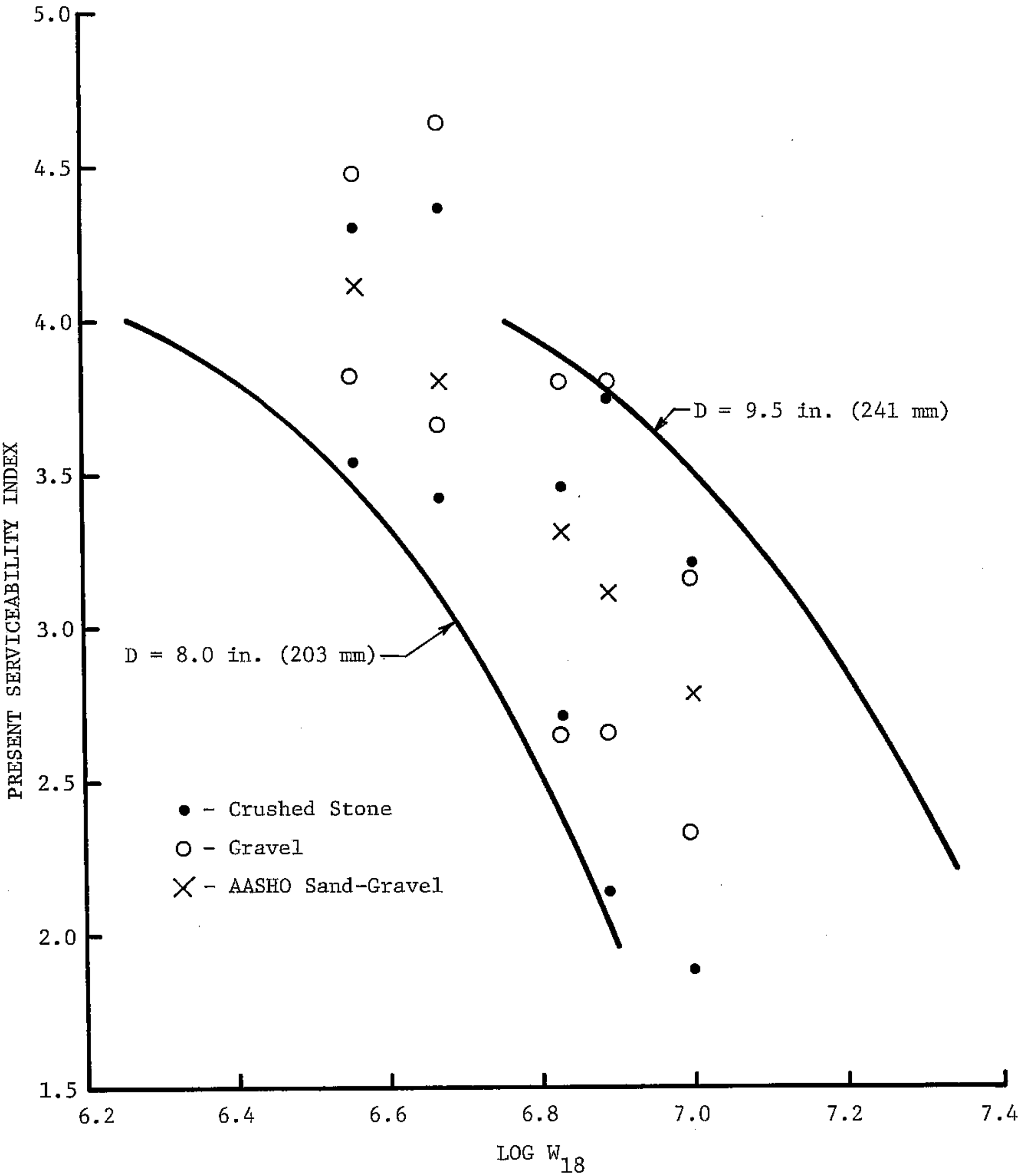


Figure 9. Performance of 8.0-in. slabs on granular subbase.

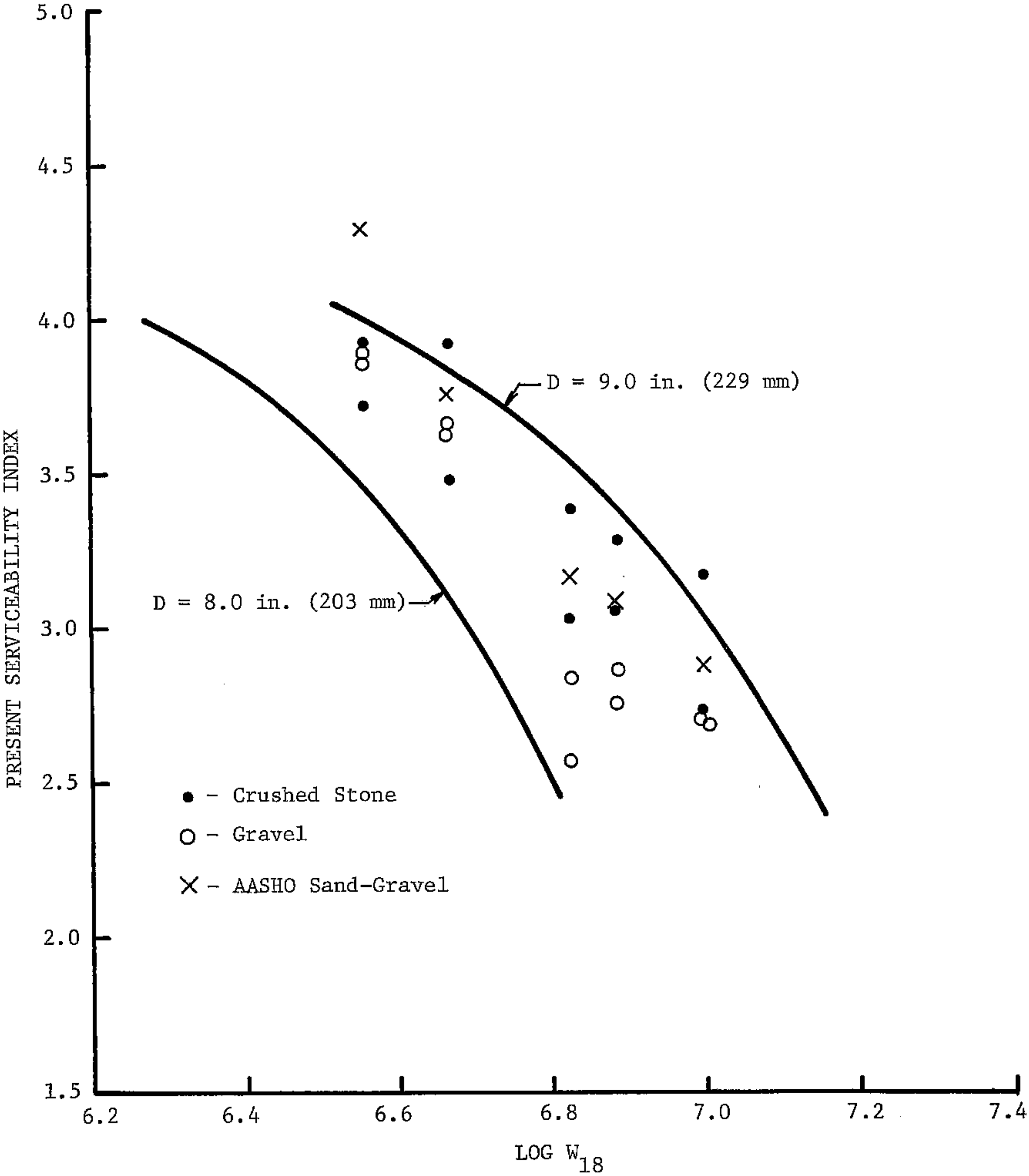


Figure 10. Performance of 9.5-in. slabs on granular subbase.

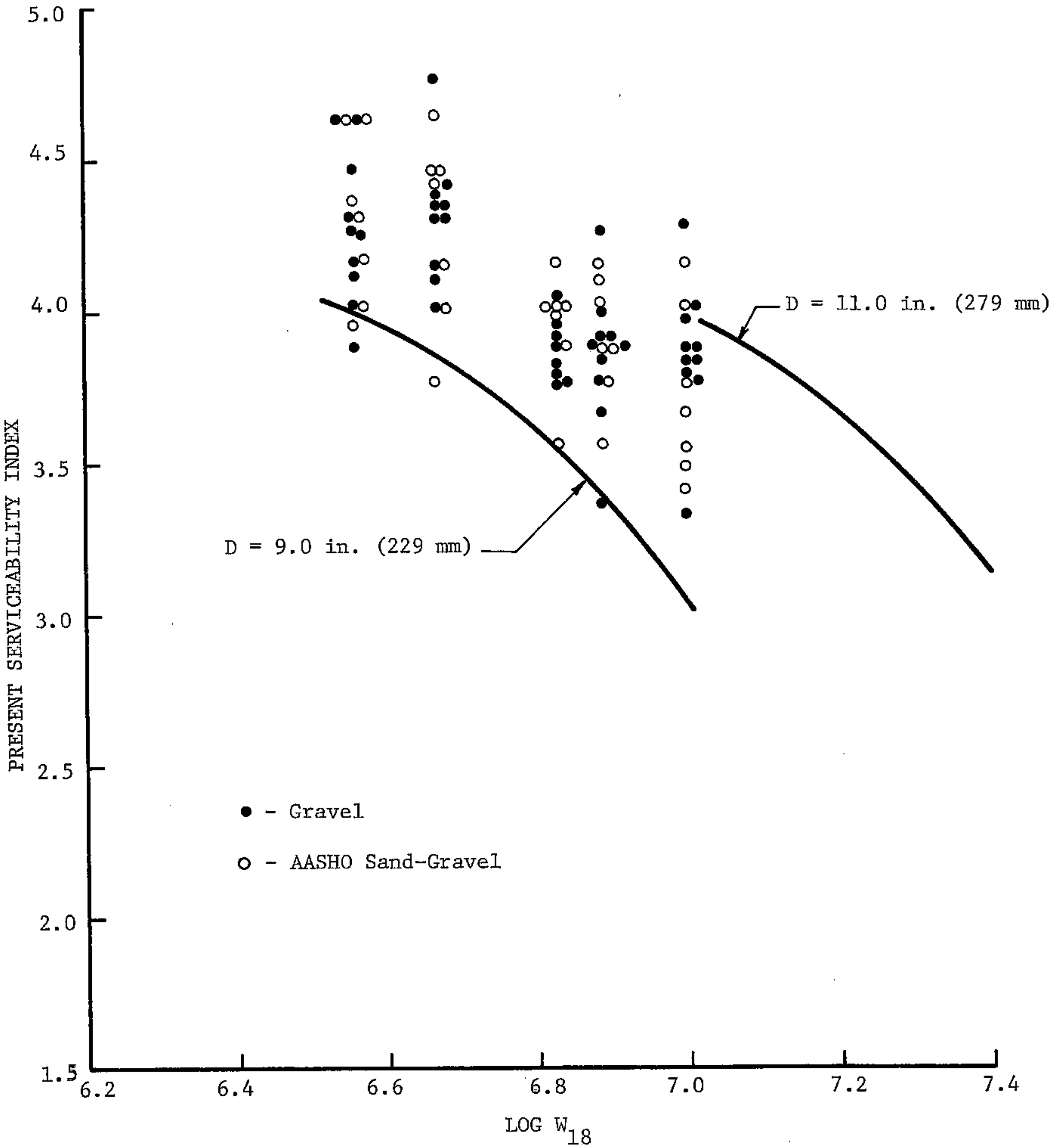


Figure 11. Performance of 10.0-in. slabs on granular subbase.

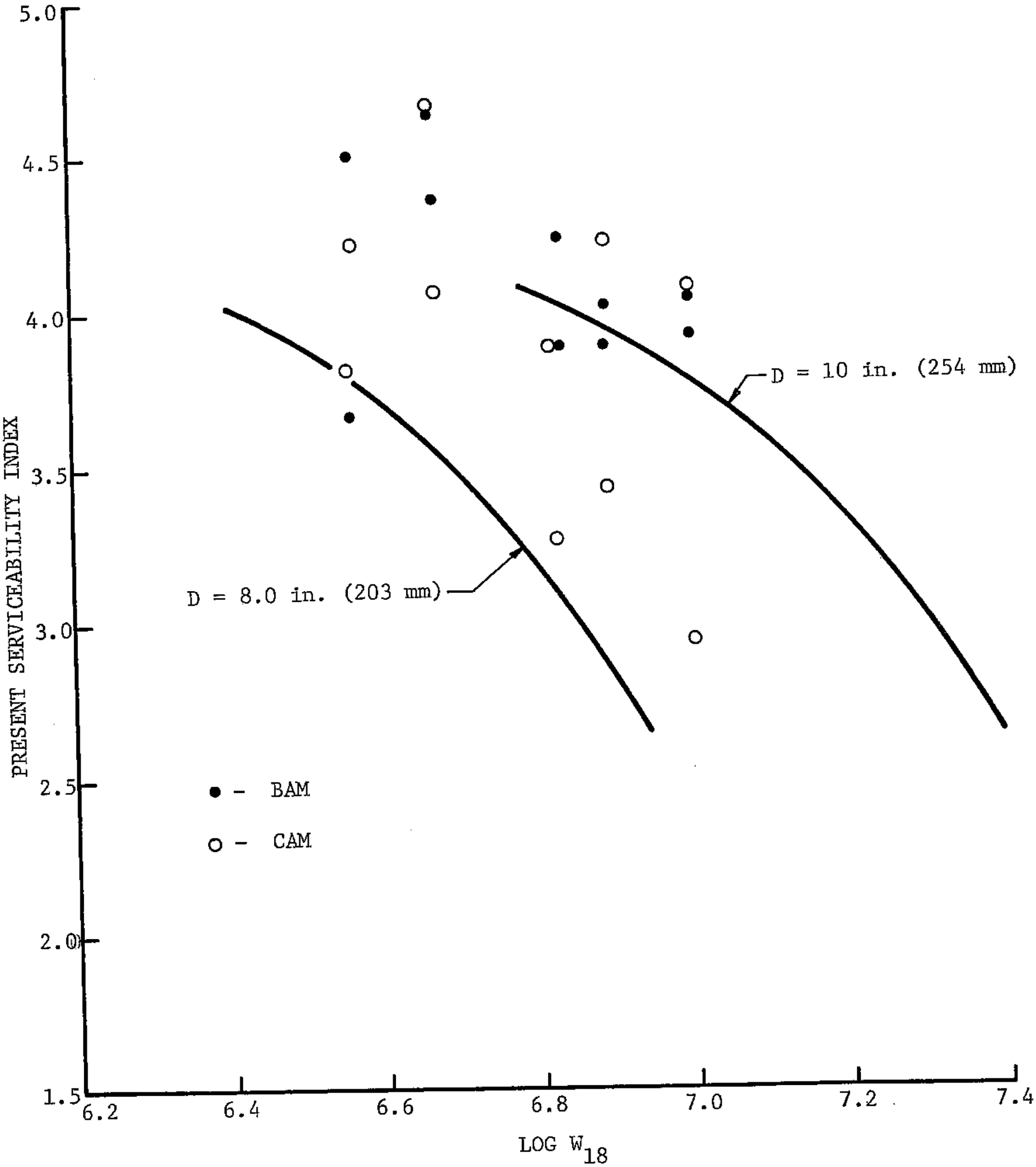


Figure 12. Performance of 8.0-in. slabs on stabilized subbase.

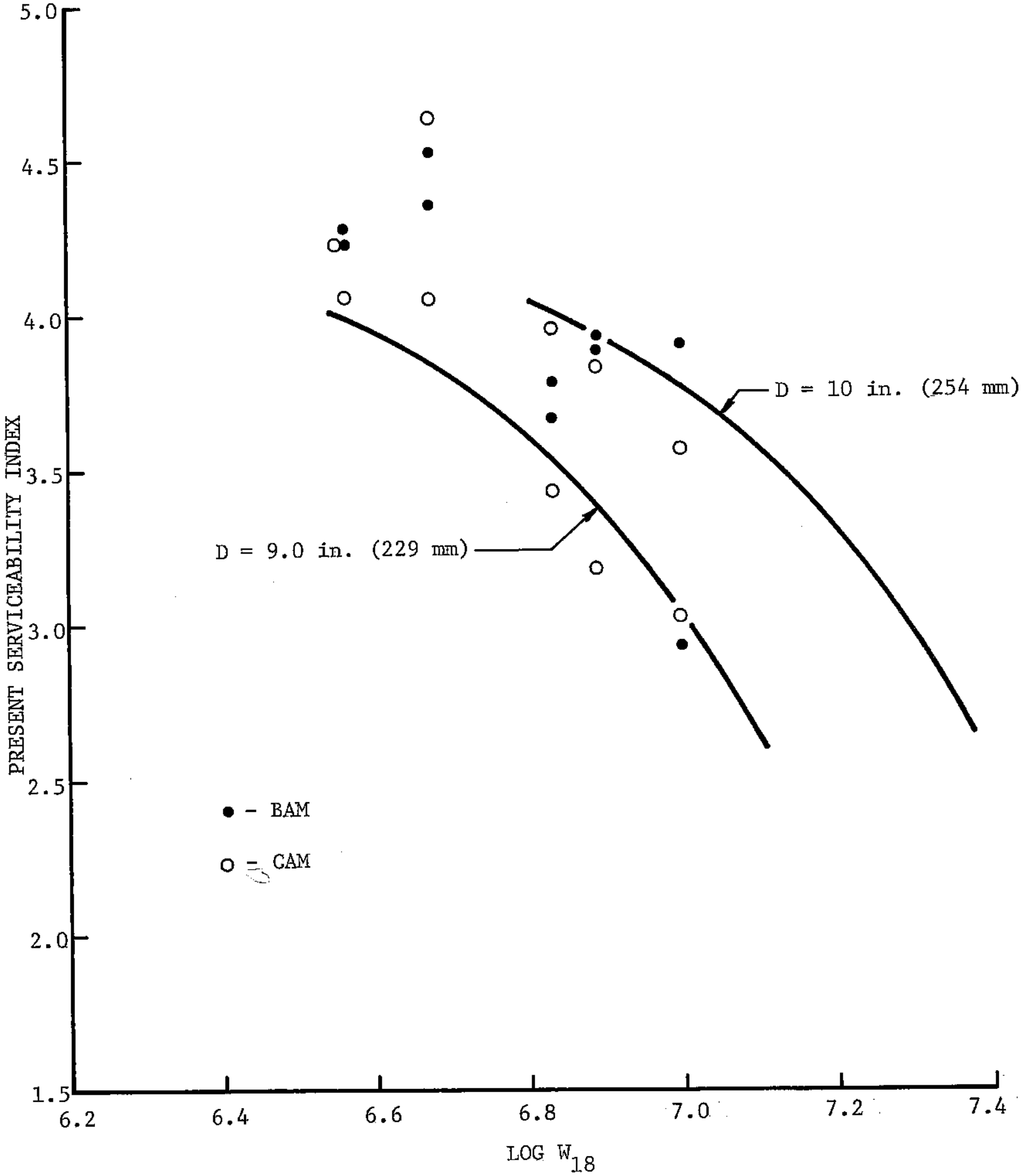


Figure 13. Performance of 9.5-in. slabs on stabilized subbase.

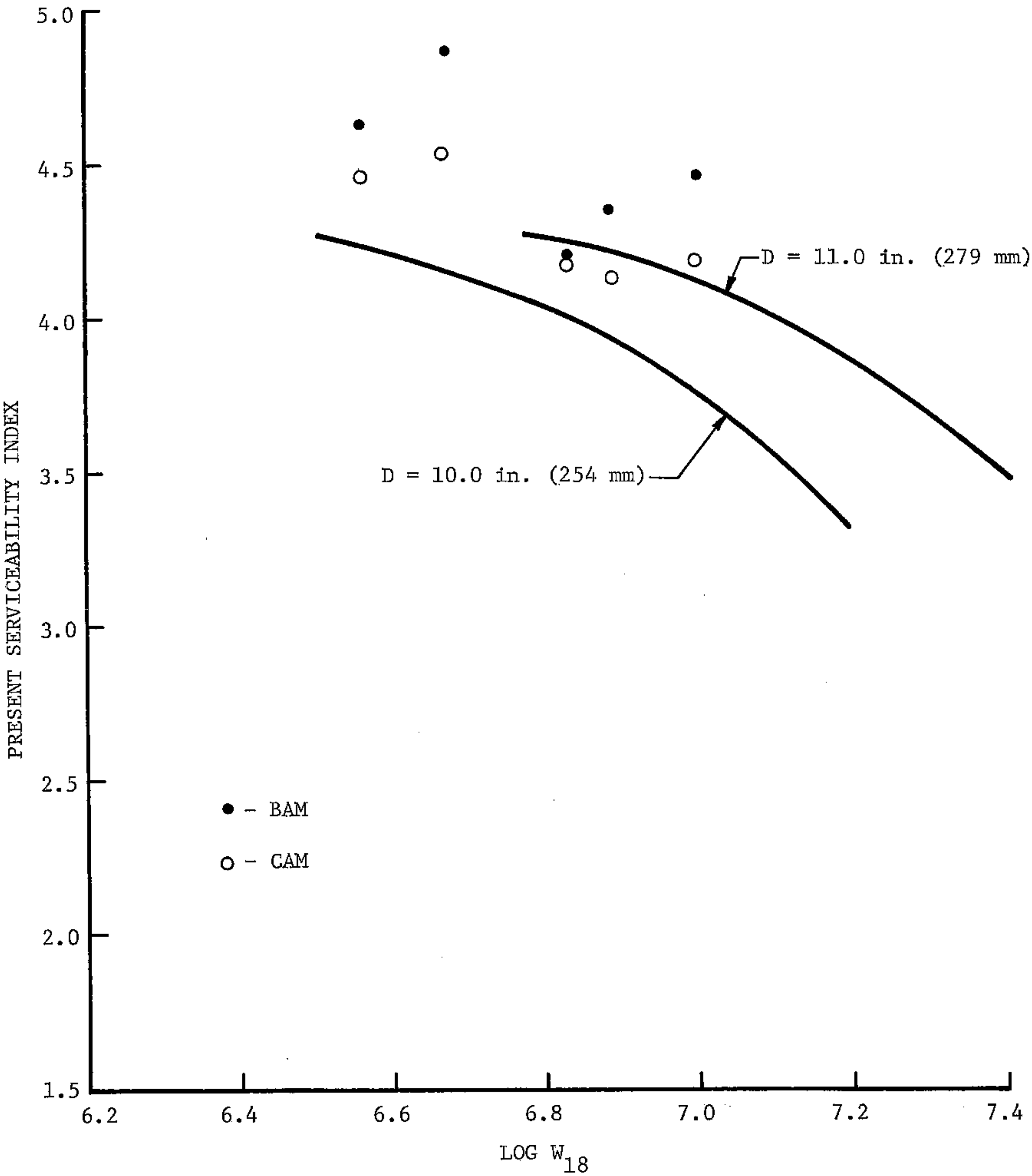


Figure 14. Performance of 10.0-in. slabs on stabilized subbase.

observed serviceability trend of the test sections, and the performance curves enclosing the points represent performance predicted by the original equation. The graph indicates that two of the 8.0-in. pavement slabs had dropped below a PSI of 2.5 (terminal for interstate highways) and that two sections remain above a PSI of 3.0. These sections performed about as would be expected for 8.5-in. Road Test pavement design.

The performance of 9.5-in. pavement sections on the granular subbases is plotted in Figure 10. The figure shows that the 9.5-in. pavement sections had a serviceability trend similar to that which would be expected of 9.0-in. AASHO Road Test design but that the terminal serviceability level had not yet been reached by 1974. In the 9.5-in. pavement sections, the performance appears slightly better on the crushed stone subbase than on the gravel.

Performance of the 10-in. pavement sections on the granular subbases is plotted in Figure 11. The serviceability trend of the new 10-in. pavement sections was about the same as would be expected of a 10-in. AASHO pavement, but the PSI had not dropped below 3.3 by 1974.

Except for the 9.5-in. pavement sections, which had developed more major cracks than the 8.0-in. and 10.0-in. pavement, the newer rigid pavement sections performed in a manner equal to or slightly better than would be predicted for them based on AASHO Road Test findings. Except for the slight difference in behavior in favor of the crushed stone subbase material that appeared in the 9.5-in. pavement sections, no difference in pavement performance among the granular subbase materials could be associated with the kind of granular material.

The performance of the pavement sections on the stabilized subbases is charted in Figures 12, 13, and 14. In Figure 12, the PSI points of the

8.0-in. pavements for 1968, 1969, 1971, 1972 and 1974 are plotted along with the AASHO performance curves. The figure shows that by 1974, the PSI of only one of the CAM test sections had fallen to 2.9, while the remaining test sections had PSI'S equal to 3.9 or higher. The performance curves indicate that the 8-in. pavements on the stabilized subbase have outperformed the AASHO equation prediction för 8.0-in. pavements and have behaved more like 9.0-in. pavements.

The performance of the 9.5-in. pavements on BAM and CAM subbases is charted in Figure 13 and the 10-in. pavements in Figure 14. Although the 9.5-in. pavements, for reasons that are unexplained, have not performed as well as the 8.0-in. pavements by AASHO Road Test standards, the performance of all pavements was greatly improved by using either BAM or CAM as subbase.

Rigid Pavement Distress

The annual condition surveys inventoried a variety of types of rigid pavement distress. The most important types were, transverse cracking, D-cracking, and spalling. Pavement blowups did not occur in any of the experimental pavement sections of the mainline pavement, but one blowup did occur near the platform scales, which is a part of the original pilot section. Another occurred in one of the test sections of loop 1. Although pavement pumping occurred in very minor amounts (traces only), measurable amounts of pumped material were never observed. Other forms of pavement distress such as scaling, corner breaks, and compression cracking were so minor in amounts that their effect on pavement behavior was not discernible.

Dowel bar corrosion is believed to be an important cause of some of the pavement distress that was observed because it resulted in the freezing of a number of the sawed, dowelled, transverse joints, particularly in the short rehabilitated test sections with 40-ft pavement panels. Freezing of the joints resulted in increased tensile stress on the reinforcing steel at pavement cracks to the point where rupture occurred, thereby allowing minor cracks to widen and become major cracks (8).

As major cracks developed they added to pavement roughness as did faulting, patching, spalling, and D-cracking, which ultimately reduced pavement serviceability to the terminal level. The amounts of transverse cracking, D-cracking, spalling, and patching that occurred in the rigid pavement sections are summarized in Tables 8, 9, 10, and 11.

A summary of the various forms of pavement distress in the non-reinforced pavements with 15-ft panels is in Table 8. As shown in the table, there was very little transverse cracking in the 15-ft panels. The largest number of cracks was in the 8-in. slabs and a few formed in the 9.5-in. slabs. There were no cracks in the 11.0-in. and 12.5-in. slabs. Due to the absence of reinforcement, almost all of the cracks quickly widened into major cracks.

The transverse cracking in the 40-ft panels is listed in Tables 9 and 10. The amounts of cracking in the AASHO test sections are in Table 9 and the amounts of cracking in the new test sections are in Table 10. In the AASHO 40-ft panels (Table 9), major transverse cracking decreased as pavement thickness increased. A similar trend in minor transverse cracking was observed in the new 40-ft panels overlying a granular subbase (Table 10). The data in Table 10 also indicate that stabilizing the subbase reduced major transverse cracking in the new 40-ft panels.

In the new 100-ft panels (Table 11) transverse cracking was greater in the pavement overlying a granular subbase than in the pavement overlying a stabilized

TABLE 8

SUMMARY OF THE TRANSVERSE CRACKING, D-CRACKING, SPALLS, AND PATCHING IN THE AASHO PAVEMENT SECTIONS WITH 15-FT PAVEMENT PANELS

Slab Thickness (in.)	Subbase Type	18-Kip ESAL Applications (Millions)	Transverse Cracking (Ft. 1000 ft^2) Minor	Transverse Cracking (Ft. 1000 ft^2) Major	D-Cracking (ft^2/1000 ft^2)	Spalls (No./1000 ft^2)	Patching (ft^2/1000 ft^2)
8	SGM	11.64	0	19.2	0.5	5.3	10.2
9.5	None	14.09	0	0	14.6	9.0	9.0
9.5	SGM	11.64	0	0	0.9	6.9	2.3
	SGM	14.09	0	2.8	18.3	2.8	5.8
	SGM	18.64	4.2	4.2	4.6	5.8	3.0
11.0	SGM	14.09	0	0	9.5	3.7	2.6
	SGM	18.64	0	0	2.9	5.0	1.3
12.5	SGM	18.64	0	0	1.6	3.9	0.9

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 kip = 453.6 kg; 1 ft^2 = 0.0929 m^2

TABLE 9

SUMMARY OF THE TRANSVERSE CRACKING, D-CRACKING, SPALLS, AND PATCHING IN THE AASHO PAVEMENT SECTIONS WITH 40-FT PAVEMENT PANELS

Slab Thickness (in.)	18-Kip ESAL (Millions)	Transverse Cracks (ft/1000 sq ft) Minor	Transverse Cracks (ft/1000 sq ft) Major	D-Cracking (ft^2/1000 ft^2)	Spalls (No./1000 ft^2)	Patching (ft^2/1000 ft^2)
8.0	11.64	36.1	40.3	0.2	2.7	7.2
9.5	11.64	20.8	29.2	0.6	3.1	8.1
	14.09	35.4	27.1	6.5	2.9	1.2
	18.64	39.6	20.8	4.2	3.5	2.1
11.0	14.09	41.7	12.5	6.3	1.0	2.0
	18.64	30.2	22.9	5.0	4.3	1.3
12.5	18.64	36.5	7.0	3.9	2.9	1.6

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 kip = 453.6 kg; 1 ft^2 = 0.0929 m^2

TABLE 10

SUMMARY OF THE TRANSVERSE CRACKING, D-CRACKING, SPALLS, AND PATCHING IN THE NEW PAVEMENT SECTIONS WITH 40-FT PAVEMENT PANELS

Slab Thickness (in.)	Subbase Type	Transverse Cracking ($ft/1000\ ft^2$) Minor	Major	D-Cracking ($ft^2/1000\ ft^2$)	Spalls (No./1000 ft^2)	Patching ($ft^2/1000\ ft^2$)
8.0	SGM	33.3	16.7	0	0.7	0.7
	Gravel	69.4	24.1	0	4.4	1.5
	Crushed Stone	37.5	22.9	0	1.7	0.2
	BAM	32.5	0	0.4	2.7	1.7
	CAM	25.0	6.3	0.7	1.6	0.5
9.5	SGM	20.8	20.8	0	2.1	0
	Gravel	5.7	25.7	0.2	1.5	0.1
	Crushed Stone	11.5	21.4	3.5	1.5	0
	BAM	19.6	10.3	0	1.6	0.6
	CAM	27.1	12.5	0	1.9	4.5

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 ft^2 = 0.0929 m^2

TABLE 11

SUMMARY OF THE TRANSVERSE CRACKING, D-CRACKING, SPALLS, AND PATCHING IN THE NEW 10-INCH PAVEMENT SECTIONS WITH 100-FT PAVEMENT PANELS

Subbase Type	Transverse Cracking ($ft/1000\ ft^2$) Minor	Major	D-Cracking ($ft^2/1000\ ft^2$)	Spalls (No./1000 ft^2)	Patching ($ft^2/1000\ ft^2$)
SGM	66.8	7.0	3.5	1.1	4.8
Gravel	56.9	1.3	2.2	1.0	0.2
BAM	16.7	0	0.8	0.9	0.2
CAM	24.8	1.9	1.2	3.8	0.2

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 ft^2 = 0.0929 m^2

subbase. The least transverse cracking occurred in the pavement on a BAM subbase. In fact, no major cracks have occurred in this pavement.

D-cracking is a series of fine, sub-parallel cracks that appear at the pavement surface along joints and cracks and at joint intersections in many pavement sections late in their service life. In the advanced stage, pavement fragments spall off in the cracked areas from both upper and lower slab surfaces. The amounts of D-cracking were greater in the older pavements than in the newer pavements.

The amount of spalling was greatest in the 15-ft panels and was least in 100-ft panels, except that the 100-ft panels on the CAM subbase had quite a few spalls. The amount of spalls per unit pavement length decreased as joint spacing increased, but the amount of spalls at each joint increased as the panel length increased.

Rigid Pavement Deflections

Static rebound deflections, using the Benkelman beam and an 18-kip single-axle load, were measured annually beginning in the fall of 1962 and in May or June of the following years. They were discontinued after the spring of 1970. The procedure that was followed was identical to that which had been introduced during the AASHO Road Test, and measurements were made at the slab edges and corners. The data have been summarized, and average pavement-edge deflections are presented in Table 12. Corner deflections differed only in being slightly larger than the edge deflections. The overall mean deflection was approximately 0.009 in. (0.23 mm). The mean deflection of each pavement slab thickness is recorded in the bottom line of the table. It can be seen that the mean pavement-edge deflection is least for the thickest pavement slabs and largest for the thinnest slabs.

TABLE 12

RIGID PAVEMENT-EDGE DEFLECTIONS BY BENKELMAN BEAM (in. x 10^{-3})

	Pavement Slab Thickness (inches)				
Year	8.0	9.5	10.0	11.0	12.5
1962	8.0	7.0	6.6	6.0	6.0
1963	12.3	11.3	9.3	9.5	9.0
1964	14.7	11.0	6.8	7.5	5.0
1965	10.0	10.0	10.3	8.5	7.0
1966	10.0	9.7	9.1	9.0	8.5
1967	10.0	9.7	10.8	9.0	8.0
1968	13.3	11.7	9.1	10.0	8.0
1970	12.0	10.3	9.0	9.0	7.0
MEAN	11.3	10.1	8.9	8.6	7.3

Note: 1 in. = 25.4 mm

Flexible Pavement Behavior

All of the flexible pavement sections in the AASHO Test Road that were structurally sound at the close of formal testing were rehabilitated by resurfacing. The new experimental sections that were added represent new designs except for two sections that duplicate AASHO designs. The resurfacing of all the original flexible test sections in the experimentation precluded a continuation of the pavement-performance studies in the flexible pavement experiments, but the test sections that have been added have provided new knowledge of flexible pavement behavior.

The flexible pavement research involves four separate studies. These are: (1) base behavior; (2) bituminous overlay behavior; (3) composite pavement behavior; and (4) behavior of test sections that were duplicates of original AASHO Design 1 sections. The base study includes three sections with granular bases. Two of these duplicated original AASHO Design 1 sections; the other is a new design. The base study also included six sections that have a bituminous aggregate mixture (BAM) base and seven sections that have a cement aggregate mixture (CAM) base. Seventeen sections in the original AASHO Design 1 experiment were retained and were resurfaced with bituminous overlays. Four test sections in the original bituminous-treated wedge-shaped base experiment also were saved and resurfaced. The composite pavement study includes six test sections with plain PCC bases.

Base Course Behavior

Base types in the flexible pavement study include bituminous aggregate mixture (BAM), cement aggregate mixture (CAM) and granular (crushed stone). The surfacing was a dense-graded bituminous concrete corresponding to Illinois Class I binder and surface course mixtures which was 4.5 in. (114 mm) thick. Base thicknesses were 8 in. (203 mm), 10 in. (254 mm), and 12 in. (305 mm) for the BAM; 10 in. (254 mm), 12 in. (304 mm), and 14 in. (356 mm) for the CAM; and 8.5 in. (216 mm) for the crushed stone. The subbase material was a dense-graded gravel corresponding to that normally specified by Illinois for gravel base and subbase construction. The subbase thickness was 4 in. (102 mm) for all stabilized base sections and 23 in. (584 mm) for the crushed stone base section.

The aggregate for the BAM base was the sand-gravel mixture salvaged from the original test sections and was mixed with 85-100 penetration grade paving asphalt. Results of extraction tests and Marshall stability tests are given in Table 13. The BAM was prepared and placed by conventional equipment. The base was constructed in 13-ft (4-m) lane widths, 1 foot (0.3 m) wider than the surface lane width, and in 2-in. (51-mm) lifts when compacted.

TABLE 13

RESULTS OF EXTRACTION AND MARSHALL STABILITY TESTS ON BAM BASE

Sieve Size	Percent Passing		
1 in.	100	Asphalt Content (%)	5.2
3/4 in.	96.5	Marshall Stability	1370
No. 4	69.6	Flow	11.0
No. 10	51.2	Air Voids (%)	1.71
No. 200	5.3		

Note: 1 in. = 25.4 mm

The aggregate for the CAM base was also the same sand-gravel material used in the original test road, except that the aggregate was salvaged. The aggregate, with 4 percent Type 1 portland cement and 8 percent moisture, was combined in a batch-type mixing plant. The CAM base was placed with an asphalt paving machine in lane widths of 13 ft (4 m) and in lifts of either 4 in. (102 mm) or 5 in. (127 mm) when compacted. When the base was completed, a rapid-curing liquid asphalt was applied at the rate of 0.2 gal per sq yd (0.0 1/m^2). Laboratory wet density of the CAM base was 148.0 pcf (2371 kg/m^3) and the mean field wet density was 150.1 pcf (2404.4 kg/m^3). The mean compressive strength was 746 psi (5144 kPa) at 7 days and 926 psi (6385 kPa) at 14 days.

The crushed limestone base corresponded to that normally specified by Illinois for Type A granular base construction. The material was adjusted to optimum moisture content in a central mix plant and placed over the subbase with a spreader box in two lifts. Each lift was compacted to no less than 100 percent of standard AASHO density (AASHO Method T-99) with a vibratory compactor.

The maximum size aggregate was 1 in. with 53 percent passing the No. 4 sieve and 11 percent passing the No. 200 sieve. The maximum dry density was 138.5 pcf (2219 kg/m^3) at an optimum moisture content of 7.1 percent. The mean percent compaction obtained in the field was 101.3 percent of optimum at a moisture content of 6.3 percent.

Data collected during the 12-year test period included rut-depth measurements, condition surveys, Roughness Index measurements and deflection measurements. From 1962 thru 1974, 6.7 million 18-kip ESAL applications had passed over each test section. These data have been used to compare base behavior.

The rut depth is the average of a series of measurements in two wheelpaths throughout the length of a section. The measurements were made with a device that was used also on the AASHO Test Road (Figure 15). The average rut depth in the test sections by type and by thickness of the base course is given in Table 14. The development of rutting with time for each base type is shown in Figure 16.

Rutting was greatest in the crushed stone base section and least in the CAM base sections. Depth of rutting was not related to base thickness in either the BAM or the CAM sections.

TABLE 14

AVERAGE DEPTH OF RUTTING IN WHEELPATHS

Base Type	Thickness (in.)	Rut Depth (in.)
BAM	8	0.51
BAM	10	0.47
BAM	12	0.50
CAM	10	0.39
CAM	12	0.36
CAM	14	0.36
CR. STONE	8.5	0.66

Note: 1 in. = 25.4 mm

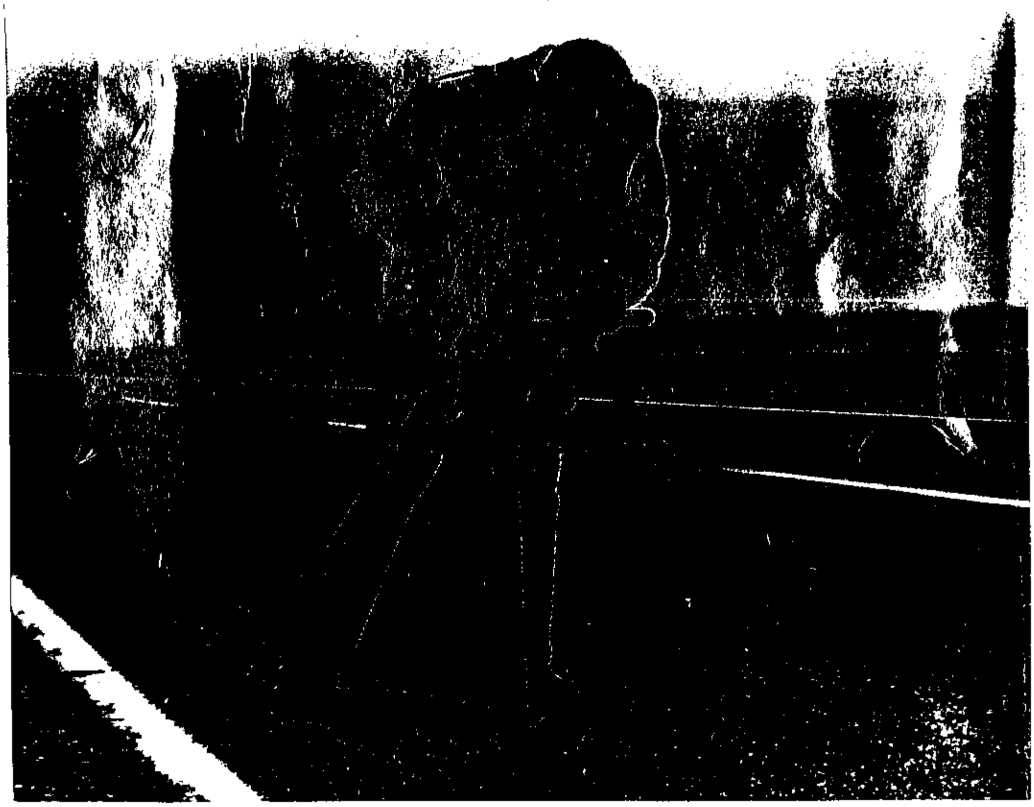
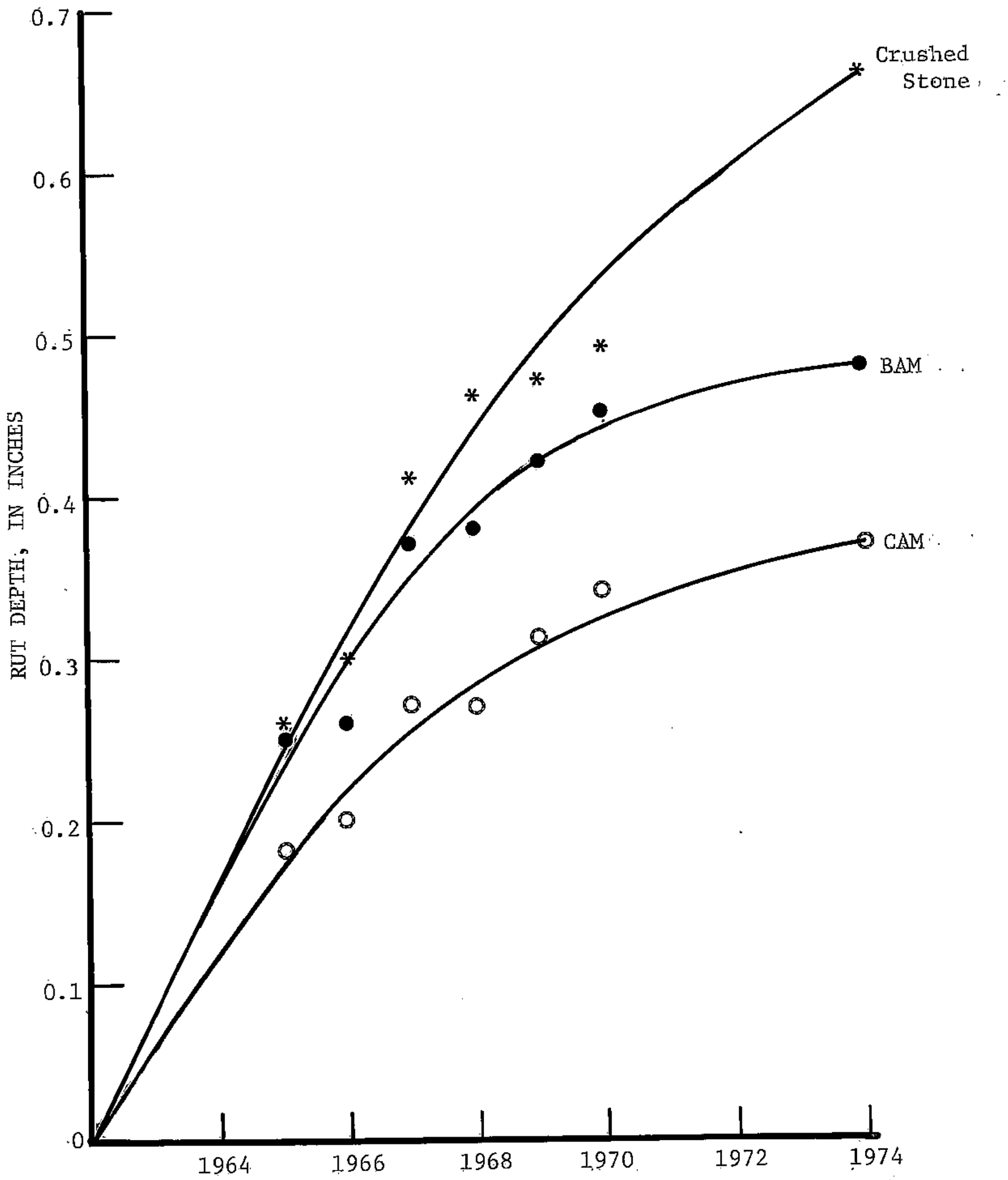


Figure 15. Manual rut-depth gage.



Note: 1 in. = 25.4 mm

Figure 16. **Development of wheelpath rutting in special base test section.**

The development of rutting with time (Figure 16) indicates that the crushed stone base section continued to rut at a high rate throughout its service life. On the other hand, ruts in the BAM and CAM base sections are developing at a lower rate than the crushed stone.

The behavior of the base test sections relative to cracking and patching is given in Table 15. Transverse and longitudinal cracking are expressed in lineal feet per 1000 sq ft of pavement surface, while area cracking (alligator-type) and patching are in square feet per 1000 sq ft of pavement. Patching was confined to skin-type patches with bituminous cold-mix.

Transverse cracking was most pronounced in the surface overlying a CAM base and least in the surface overlying a crushed stone base. It is believed that the large amount of cracking in the surface on the CAM base resulted from shrinkage cracks that formed in the CAM base during curing. Transverse cracking decreased as base thickness increased. Most of the transverse cracks in the surface on the CAM base became "tented" (surface heaving at cracks) during freezing weather, and adversely affected pavement riding quality. Tenting was not observed in the surface overlying any other base types. Longitudinal cracking was very slight for all base types.

Area cracking was most pronounced in the surface on the crushed stone base, intermediate on the CAM, and least on the BAM; likewise, patching of the surface was the greatest on the crushed stone, intermediate on the CAM, and least on the BAM. Both area cracking and patching were almost completely absent in the BAM sections. Overall structural distress, as indicated by cracking and patching, was greatest on the crushed stone base and least on the BAM base.

TABLE 15

AVERAGE AMOUNTS OF CRACKING AND PATCHING IN THE FLEXIBLE PAVEMENTS BY TYPE OF BASE

Base Type	Thickness (in.)	Transverse (Lin. ft/ 1000 sq ft)	Longitudinal (Lin. ft/ 1000 sq ft)	Area Cracking (Sq ft/ 1000 sq ft)	Skin Patching (Sq ft/ 1000 sq ft)
BAM	8	23	0	1.5	0
BAM	10	11	1	1.6	1.6
BAM	12	26	0	0	0
CAM	10	52	10	24	4.5
CAM	12	41	13	11	27
CAM	14	33	0	2.8	0
CR. STONE	8.5	13	0	246	42

Note: 1 in. = 25.4 mm
$1\ ft^2 = 0.0929\ m^2$

The Roughness Index and the Present Serviceability Index have been widely used in Illinois as an aid in evaluating the quality of new pavement construction and for determining the need for rehabilitating old pavements (2). The values have been grouped and adjective rating terms have been assigned to the groups for statewide use as shown below.

Rigid Pavement		Flexible Pavement		Descriptive Rating
RI	PSI*	RI	PSI*	
75	4.00	60	3.98	Very Smooth
90	3.66	75	3.60	Smooth
125	3.05	105	3.03	Slightly Routh
170	2.48	145	2.48	Rough
220	2.00	190	2.02	Very Rough
375	1.01	330	1.09	Unsatisfactory

* The PSI values are based on RI only.

The mean RI and PSI values for flexible pavements with BAM, CAM, and Crushed Stone base courses are shown in Tables 16 and 17. As can be seen, the BAM base sections were in the best condition. Riding quality of the pavement would be rated as "smooth" to "very smooth," indicative of a high level of service as late as 1974. The CAM base sections, however, were providing a rough ride in 1974 and the PSI'S had reached a level indicating a need for rehabilitation. The crushed stone base pavement, while having a reasonably low RI (74), developed more rutting and structural distress by 1974 than the other designs, and its PSI (2.9) was approaching the terminal value of 2.5.

TABLE 16

MEAN ROUGHNESS INDEX VALUES FOR FLEXIBLE PAVEMENTS BY TYPE OF BASE COURSE

Base Type	Thickness (in.)	1962	1964	1965	1966	1968	1969	1971	1972	1974
BAM	8	69	42	65	58	52	45	57	68	63
BAM	10	62	46	63	47	41	33	41	41	40
BAM	12	56	35	63	39	33	33	33	33	33
CAM	10	70	47	81	77	101	96	128	140	154
CAM	12	65	45	69	71	76	80	108	112	127
CAM	14	60	46	65	66	95	67	106	117	127
CR. STONE	8.5	59	40	64	47	48	38	55	60	74

Note: 1 in. = 25.4 mm

TABLE 17

MEAN PRESENT SERVICEABILITY INDEX VALUES FOR FLEXIBLE PAVEMENTS BY TYPE OF BASE COURSE

Base Type	Thickness (in.)	1962	1964	1965	1966	1968	1969	1971	1972	1974
BAM	8	3.7	4.6	3.7	3.9	4.1	4.2	3.8	3.4	3.6
BAM	10	3.9	4.4	3.8	4.3	4.2	4.8	4.4	4.1	4.2
BAM	12	4.1	4.9	3.7	4.8	4.8	4.7	4.7	4.7	4.7
CAM	10	3.7	4.4	3.5	3.5	3.1	3.1	2.6	2.5	2.2
CAM	12	3.8	4.4	3.6	3.6	3.4	3.3	2.7	2.7	2.5
CAM	14	4.0	4.4	3.8	3.5	3.1	3.6	2.9	2.7	2.4
CR. STONE	8.5	4.0	4.7	3.8	4.3	4.0	4.4	3.8	3.6	2.9

Note: 1 in. = 25.4 mm

Although the pavements with the BAM base were smooth and were rated high in serviceability by AASHO Road Test standards, they had developed ruts that averaged approximately 0.5 in. (13 mm) deep (Table 14), which is intermediate between the sections with the CAM base and those with the crushed stone base. Ruts of this magnitude were observed to cause difficulty for drivers in maintaining a straight course relative to the wheelpaths in the road. The distance between ruts in the pavement did not match the track width of passenger car wheels, and so passenger cars tended to wander in and out of the ruts. The tendency was so noticeable that on numerous occasions the drivers were observed stopped on the shoulder to examine their vehicles. During rainstorms, rainwater accumulates in the ruts, thereby producing extremely difficult driving conditions at normal highway speeds and causing some vehicle operators to straddle the inner wheelpath of the outer driving lane.

Creep-speed rebound deflections, measured with a Benkelman beam and an 18-kip single-axle load, from 1962 to 1970 showed very little change as can be seen in Table 18. Deflections were smallest on the CAM base, intermediate on the BAM, and greatest on the crushed stone base. On the BAM and CAM base, deflections for the most part, decreased as base thickness increased.

TABLE 18

AVERAGE REBOUND DEFLECTIONS BY BASE COURSE TYPE

Base Type	Thickness (in.)	Spring Rebound Deflection (in. x 10^{-3})						
		1963	1964	1965	1966	1967	1968	1970
BAM	8	26	25	16	19	15	16	17
BAM	10	15	18	13	14	11	11	13
BAM	12	15	15	13	18	12	16	14
CAM	10	10	10	10	11	13	13	13
CAM	12	10	9	9	10	11	10	14
CAM	14	6	6	9	8	10	10	10
CR. STONE	8.5	22	22	12	17	18	20	22

Note: 1 in. = 25.4 mm

In 1973 when plans were being prepared to overlay the experimental pavements, provision was made to open distressed areas and to examine the extent of rutting by trenching the pavement.

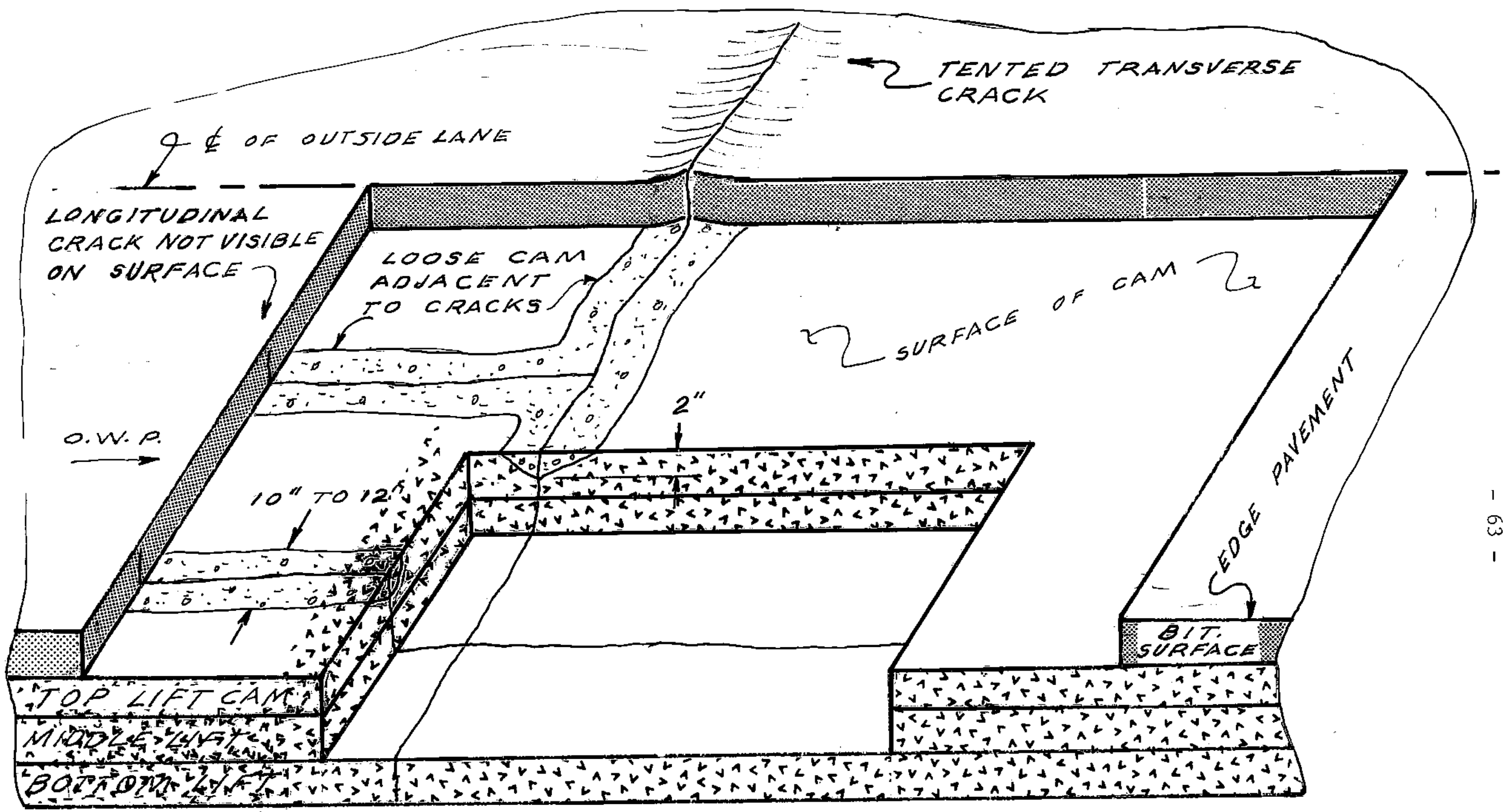
Rutting was studied in the crushed stone base section which had a 4.5-in. surface, an 8.5-in. crushed stone base, and a 23-in. gravel subbase. A trench, 6 ft (1.8 m) long and 4 ft (1.2 m) wide, was excavated to the surface of the embankment soil across the outside wheelpath in the outer lane. The normal 4-ft rut-depth gage indicated a rut depth of 0.66 in. (17 mm) but the gage was too narrow to span the entire rut. A 6-ft straightedge, which spanned the rut, indicated a rut depth of 0.88 in. (22 mm). Measurements, made as each layer was removed, indicated that the rutting occurred primarily in the surface and in the base. The rut depth was approximately 0.5 in. (13 mm) in the base, was barely detectable in the subbase, and was absent in the embankment. Density tests and thickness measurements of the base and of the subbase were inconclusive--that part of a rut resulting from either consolidation or displacement of the material could not be determined. AASHO Road Test data, however, indicated that pavement rutting was due principally to decreases in thickness of the component layers. Inspection at the AASHO Road Test found that "a rut could be attributed to changes in thickness of 32 percent, 14 percent and 45 percent, respectively, in surfacing, base, and subbase, and to a rut in the embankment soil equal to 9 percent of the total rut." Part of this change in thickness was attributed to an increase in density (4).

Transverse humps accompanied by transverse cracks on either side appeared early in the life of some of the experimental CAM base sections. The humps extended across the pavement from edge to edge and were approximately 3 ft (0.9 m) wide and 0.75 in. (19 mm) high. During the winter the cracks became tented and, along with the humps, caused a very rough riding pavement. By late summer the tented cracks were ironed out by traffic in the wheelpaths but the humps remained. Cutaway drawings (Figures 17 and 18) illustrate the observed conditions.

To investigate this distress, the outer half of the surface in the traffic lane was removed, for a length of 20 ft (6.1 m) at two locations and 8 ft (2.4 m) at a third location, exposing the surface of the CAM base course where the humps occurred.

At all three places the humps were associated with a feathered joint in the top lift of the CAM base course. At two locations CAM apparently had been placed by hand in the depression at the feathered joint to bring the top lift up to grade. At the third location there was no depression. The feathered joints corresponded with an interruption in construction such as at the end of a day's work. An overriding movement of the top CAM layer caused the surface to heave, forming the hump. Specifications for CAM base in Illinois have required vertical faces at transverse construction joints for a number of years.

At two excavations the top two lifts of CAM had extensive deterioration. Much of the CAM had disintegrated into fragments resembling the original aggregate. At the third location the CAM was in better condition, and the cracks in the surface could be traced through all three lifts. In the top lift at the third location, disintegration of the CAM was confined to an area 5 to 6 in. (127 to 152 mm) wide on each side of the cracks and 2 in. (51 mm) deep. Frost heave of the fragmented material is regarded as the cause of tenting at the cracks. At one location longitudinal cracks were observed in the CAM on both sides of the wheelpath. Although not visible in the top of the wearing surface, these cracks had reflected into the bottom course at the wearing surface.



Note: 1 in. = 25.4 mm

Figure 17. Cutaway view of a tented transverse crack in a CAM base section.

At two of the excavations the surface of the CAM was damaged and rut-depth measurements could not be obtained; however, at the third location, measurements were obtained using a 6-ft straightedge. Rut depth in the bituminous surface was 0.62 in. (16 mm), in the top lift of CAM it was 0.31 in. (8 mm), and in the middle lift it was 0.19 in. (5 mm). The presence of a rut in the lower CAM lift without the presence of longitudinal cracking leaves unanswered the question of how the rut formed in the CAM. In all cases, the CAM base east of the hump was in worse condition than it was west of the hump.

Bituminous Overlay Behavior

The overlay study includes 17 original test sections from the AASHO Flexible Design 1 experiment that have a granular base and subbase and four sections from the AASHO Flexible Design 4 experiment that have a BAM base that is wedge-shaped--the base decreases uniformly in thickness in the direction of traffic. The flexible test sections were deeply rutted during the AASHO Road Test and were rehabilitated by resurfacing. The resurfacing includes a levelling binder course and binder courses which were placed to within 3 in. (76 mm) of the proposed grade. The final binder course and the surface course were each uniformly 1 1/2 in. (38 mm) thick.

Four separate inert mineral fillers were used in the surface course. They were: limestone dust, hydrated lime, pulverized kaolin clay and asbestos fiber. The limestone dust, hydrated lime, and kaolin clay were used individually in the mixtures. Limestone dust, which is the standard mineral filler in Illinois, was used in combination with the asbestos fiber. For equivalent work-ability, it was necessary to increase the asphalt content of the mix containing asbestos fiber by 1.2 percent (10).

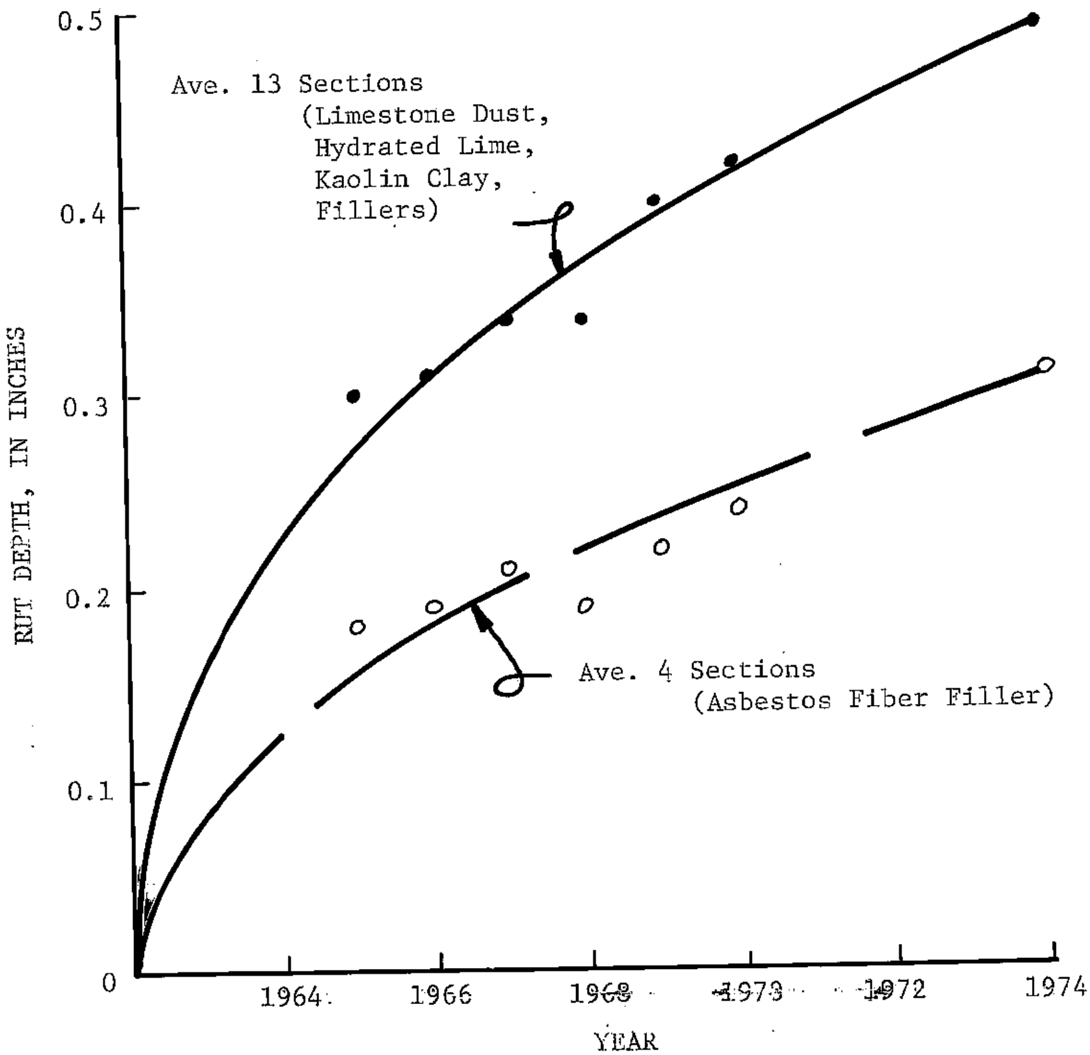
As seen in Table 2 and Table 4, the overall total structural design of the overlaid sections is quite varied. The sections from the AASHO Design 1 experiment have a total structural thickness range from 23 in. (584 mm) to 36 in. (914 mm) which was determined by coring the wheelpath in the completed roadway. The four sections from the AASHO Design 4 experiment have a similar range in structural thickness from 18 in. (457 mm) at the thinnest end to 31.5 in. (800 mm) at the thickest end. Although the structural thickness covered a wide range, there was no difference in the RI or PSI that could be associated with the thickness variation. All 21 overlaid sections tend to follow the same pattern of decreasing RI and increasing PSI with increasing numbers of load application. For this reason the RI'S and PSI'S for the resurfaced sections were averaged for each year and the values along with the $LogW_{18}$ are presented in the table below.

AVERAGE OF ALL 21 SECTIONS

Year	PSI	RI	$LogW_{18}$
1962	3.8	66	5.217
1964	4.5	46	5.642
1965	3.7	68	5.927
1966	4.5	42	6.105
1968	4.7	37	6.421
1969	4.7	35	6.517
1971	4.5	40	6.667
1972	4.5	41	6.732
1974	4.5	37	6.828

As can be seen in the table, the Roughness Index decreased, indicating a smoother riding pavement but the wheelpaths became more deeply rutted.

Rut depth data for the 17 overlaid sections from AASHO Design 1 are charted in Figure 19. As can be seen in the figure, sections containing



Note: 1 in. = 25.4 mm

Figure 19, Development of wheelpath rutting in 17 overlaid sections.

asbestos fiber filler, on the average, rutted less than the other three filler types. The range in rut depth of the sections containing limestone dust, hydrated lime, and kaolin clay as mineral filler in 1974 was from 0.46 in. (12 mm) to 0.53 in. (14 mm) while the range of the asbestos fiber filler sections was 0.27 in. (7 mm) to 0.37 in. (9 mm). The four wedge-shaped base sections (AASHO Design 4) had rut depths that were within 0.05 in. (1 mm) of the rut depth of the 13 sections whose surfaces contain limestone dust, hydrated lime, and kaolin clay.

Cracking in the overlaid pavement sections was minor. Most of the cracks that formed were short transverse cracks that resembled slippage cracks. On the average they amounted to 14 lineal ft of cracking per 1000 sq ft (4.3 m/92.9 m^2) of pavement. Area cracking (alligator-type) amounted to only 0.9 sq ft per 1000 sq ft (0.08 m^2/92.9 m^2) of pavement, and longitudinal cracking amounted to only 0.8 ft per 1000 sq ft (0.2 m/92.9 m^2) of pavement.

Wheelpath rebound deflections were low. They remained similar among test sections throughout the study period although they varied from year to year. The mean wheelpath deflection varied between a low of 0.009 in. (0.23 mm) in 1962 to a high of 0.014 in. (0.36 mm) in 1964.

The behavior of the overlaid sections indicated that they were sufficiently strong to carry Interstate traffic without developing any more than minor amounts of structural distress.

Composite Pavement Behavior

The flexible pavement tangent includes six composite test sections, 400 ft (121.92 m) long, which were placed under bridges that carry local traffic over the Interstate highway. Each test section has a nonreinforced, unjointed, portland cement concrete base course of paving-grade concrete that is either

8.0 in. (203 mm) or 9.5 in. (241 mm) thick, and the surface course is bituminous concrete 3.0 in. (76 mm) thick. Under the bridges, the elevation of the subgrade had to be lowered to accommodate the pavement while providing the necessary clearance for highway traffic. The subbase therefore varied in thickness and was a mixture of the AASHO sand-gravel material and crushed stone. Two of the composite sections were surfaced with the bituminous mix containing asbestos fiber and four were surfaced with the standard surface mix.

The behavior of the composite pavement sections is summarized in the table below. As can be seen in the table, the depth of rutting was low. The mean rut depth in the surface with standard filler was 0.27 in. (7 mm) as compared with 0.17 in. (4 mm) for the surface containing asbestos fiber. This reduction is similar to that found in the overlaid sections.

Measurement	Value
Mean Rut Depth (in.)	
(a) Standard filler	0.27
(b) Asbestos filler	0.17
Cracking (linear ft per 1000 sq ft	20.4
Rebound Edge Deflections (in.)	0.010
Roughness Index (Average 1974 8.0" Base)	74.0
Roughness Index (Average 1974 9.5" Base)	74.0
Present Serviceability Index (Average 1974 8.0" Base)	3.4
Present Serviceability Index (Average 1974 9.5" Base)	3.4

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 ft^2 = 0.0929 m^2

Of the cracks that formed in the surface, all but one were reflected cracks that formed in the PCC base before it was overlaid.

The mean rebound edge deflection was 0.010 in. (0.25 mm) for both the 8.0-in. (203-mm) and 9.5-in. (241-mm) bases.

Duplicate AASHO Design Behavior

Two test sections that were built in 1962 were duplicates of original AASHO Designs. One section had a 4-in. surface of Illinois Standard Class I bituminous concrete, and the other had a 5-in. surface. The base courses were 6 in. (152 mm) of salvaged AASHO Road test crushed stone, and the subbases were 12 in. (305 mm) of salvaged AASHO Road Test sand-gravel material. The materials are described elsewhere (1, 3).

			Pavement Design			
			4-6-12		5-6-12	
Measure	Year	$LogW_{18}$	RI	PSI	RI	PSI
Performance	1962	5.217	76	3.6	80	3.5
	1969	6.517	80	3.3	38	4.6
	1972	6.732	84	3.2	76	3.4
	1974	6.828	144	1.7	110	2.4
Cracking (Sq ft)			538		67	
Patching (Sq ft)			80		0	
Deflection Range (in.)			0.016 - 0.023		0.012 - 0.020	

Note: 1 in. = 25.4 mm 1 ft^2 = 0.0929 m^2

The behavior of the two duplicate test sections is described by the data in the above table. As can be seen in the table, the Roughness Index increased and the level of serviceability decreased as load applications ($LogW_{18}$) increased until 1974 when the test sections dropped below a PSI of 2.5, which is terminal for interstate highways in Illinois. Both test sections had developed much area cracking with the 4-6-12 design having the greater amount.

In the AASHO Test Road six sections were of the 4-6-12 design, and eight sections were of the 5-6-12 design. These sections were in the three heavier loops in both the single-axle and the tandem-axle load lanes. As a means for comparison with the AASHO sections, the applications observed at the AASHO Test

Road for each section that reached a PSI of 2.5 were converted to 18-kip ESAL applications ($LogW_{18}$), and their values are given below. One of the AASHO Test Road sections in the 4-6-12 design and two in the 5-6-12 design remained above a PSI of 2.5 during the AASHO tests and are not shown in the table.

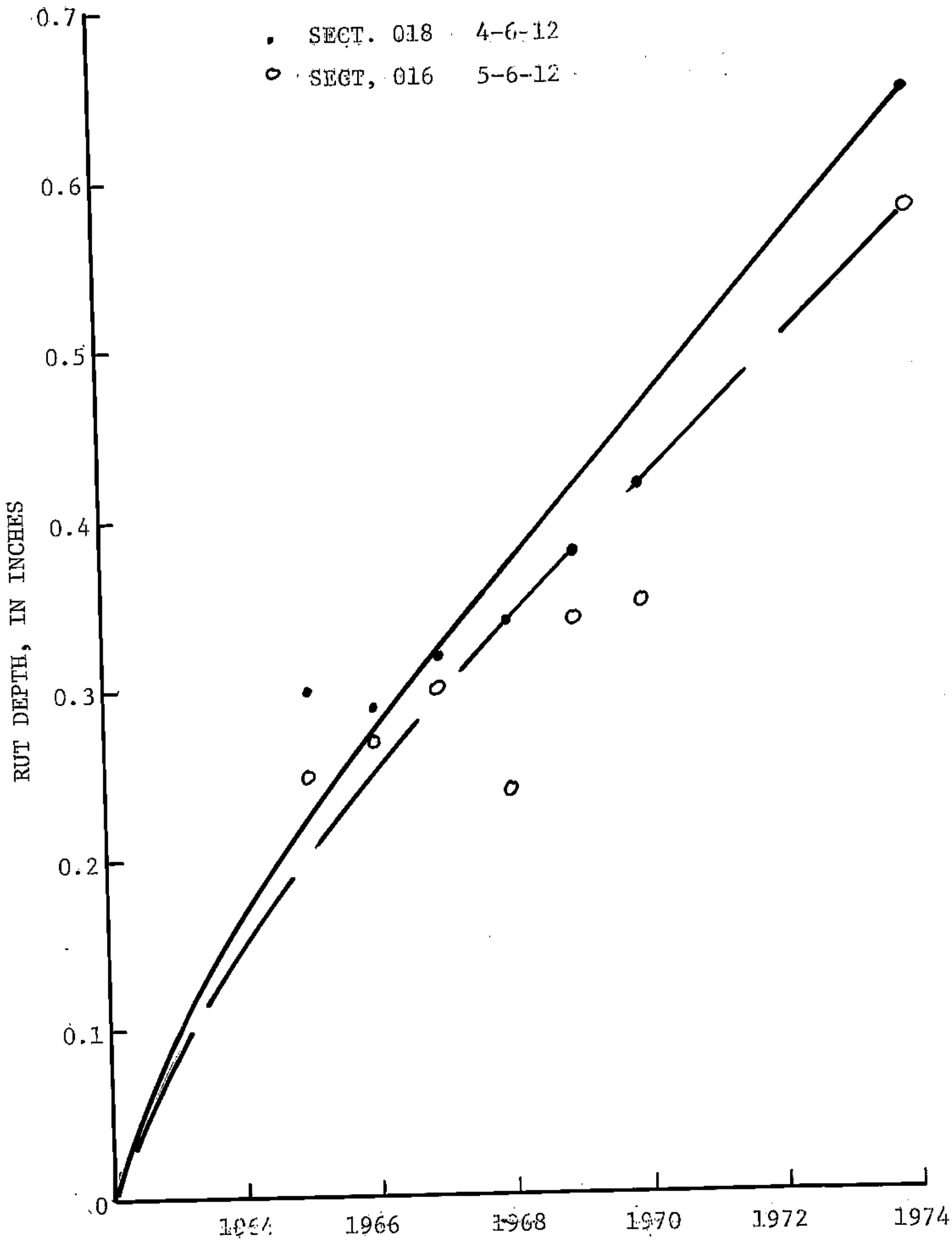
$LogW_{18}$ AT PSI = 2.5 FOR THE AASHO ROAD TEST SECTIONS AND DUPLICATE SECTIONS

4-6-12		5-6-12
	AASHO Sections	
5.891		6.284
6.010		6.269
5.697		6.604
5.759		6.102
5.680		6.271
		6.436
	Duplicate Sections	
6.770		6.810

As can be seen in the table the duplicate test sections sustained more 18-kip ESAL applications than the AASHO sections to a PSI of 2.5.

From the analysis of residual variation at the AASHO Road Test (4) it was found that a deviation of $\pm$ 0.46 LogW would be tolerable for nine chances out of ten that the duplicate section has the same performance as the AASHO sections. The $LogW_{18}$ values obtained by the duplicate (4-6-12) test section are outside those limits and the 5-6-12 duplicate is at the limit. Therefore, there is less than a 90 percent chance that the duplicate test sections have the same performance as the AASHO sections.

The formation of wheelpath ruts followed a normal trend, as can be seen in Figure 20. The difference in rut depth between the two sections is attributed to the thickness of bituminous concrete surfacing. Rutting in the duplicate sections was similar in magnitude with AASHO Test Road sections of the same design.



Note: 1 in. = 25.4 mm

Figure 20. Development of wheelpath rutting in duplicate sections.

The wheelpath rebound deflections of the duplicate test sections were similar to those in other designs, and the variation from year to year was minimal. The test section with 4-in. surfacing had a range in deflection from 0.016 in. (0.41 mm) to 0.023 in. (0.58 mm), and the 5-in. section had a range in deflection from 0.012 in. (0.30 mm) to 0.020 in. (0.51 mm).

DISCUSSION

The post Road Test research, conducted at Ottawa, Illinois, was an extension of the AASHO Road Test pavement sections under mixed interstate traffic. At the same time, new structural materials that had not been used in the original Road Test were added to the experimentation. These were Illinois gravel and crushed stone as granular materials and BAM and CAM as stabilized aggregate mixtures. The gravel was used as subbase in both rigid and flexible pavement sections; the crushed stone was used as a base course in new flexible pavement designs and as a subbase in new rigid pavement designs. The stabilized aggregate mixtures were used both as the subbase under portland cement concrete pavements and as the base course under bituminous concrete pavements.

All of the original portland cement concrete pavement sections that were capable of carrying interstate traffic for a reasonably long service life became a part of the new test facility in 1962 when new portland cement concrete pavements were added. All of the original bituminous concrete pavement sections that were retained in the new facility were resurfaced with bituminous concrete overlays to restore their serviceability. New flexible pavement sections with stabilized base (BAM and CAM); granular base (crushed stone), and composite sections (Plain PCC base) were built. Also, four sections, two rigid and two flexible, duplicated original AASHO designs.

Including new pavement designs and new materials to the experimentation added a new dimension to the post Road Test Research. It became possible not only to evaluate the performance of original AASHO Road Test sections under regular highway traffic conditions and to test the performance equations, but also to make comparisons with new types of materials and new pavement designs in the environment of the original Road Test. As a first step in the evaluation process, it was necessary to reconcile the effects of the mixed and variable loadings of the regular highway traffic with the controlled loadings of the AASHO Road Test so that the pavement serviceability-performance concept could be applied in the evaluation of pavement behavior. This was accomplished by computing load equivalency factors from the Road Test equations. The equivalency factors permitted the adjustment of all loadings to equivalent 18-kip single-axle loads. The 18-kip single-axle load was selected for this purpose because it was the maximum legal single-axle weight that was permitted for primary highways in many of the states including Illinois. The logarithm of the cumulative number of applications of equivalent 18-kip single-axle loads ($LogW_{18}$) in relation to the Present Serviceability Index gave a serviceability trend (performance curve) which was useful to evaluate pavement behavior.

As a next step in the pavement performance research, the behavior of the original AASHO rigid pavement sections in the rehabilitated test road was evaluated in relation to the performance indicated for them by the AASHO Road Test performance equation. The findings were that the performance equation had predicted the behavior of the 8.0-in. (203-mm) and 9.5-in. (241-mm) rigid pavement sections reasonably well, but that it tended to predict better performance for the 11.0-in. (279-mm) and 12.5-in. (318-mm) rigid pavement sections than had been actually observed (Figures 4, 5, 6, and 7). In view of the findings, it was decided to modify the performance equation for rigid pavements so that it would do a better job of predicting the performance of the thicker pavement sections. For this purpose, the data from the rehabilitation study were reinforced with all data from the original Design 1 rigid pavement experiment that were useable, and by multiple regression techniques the following modified equation was obtained:

$$\mathrm{Log}W_{18} = 2.724 + 4.5 \mathrm{Log}(D_2 + 1) + G$$

Figure 21 shows the new performance curve computed from the modified equation for each slab thickness variation in the test in relation to the last PSI measurements (1974) prior to resurfacing. As an adjunct to the data for the 8.0-in. pavements, the PSI values for 1972 have been added to the chart.

As can be seen in Figures 8 and 21, the modified equation increases the performance expectation for the 8.0-in. pavements slightly but decreases the expectation for the 9.5-in. pavements in relation to the original equation. The expectation for the 11.0-in. and 12.5-in. pavements also is decreased by an ever-increasing amount as the pavement thickness increases. As can be seen in Figure 21, the curves for the thicker pavement slabs fit the 1974 data about as well as the AASHO curves fitted the original data. Analysis of the residuals gave a root mean square residual for $\mathrm{Log}W_{18}$ of 0.21 for the modified equation while the root mean square residual for LogW was 0.22 in the original analysis.

The experimental subbases (Figures 9 to 14) by the end of 1974 had carried the equivalent of 10 million 18-kip single-axle loads. By that time the 8-in. pavements on granular subbases (Figure 9) were nearing the terminal serviceability point for interstate highways but the 9.5-in. pavements and the 10-in. pavements were in better condition. There was no difference in performance in any of the thickness variations that could be associated with the type of

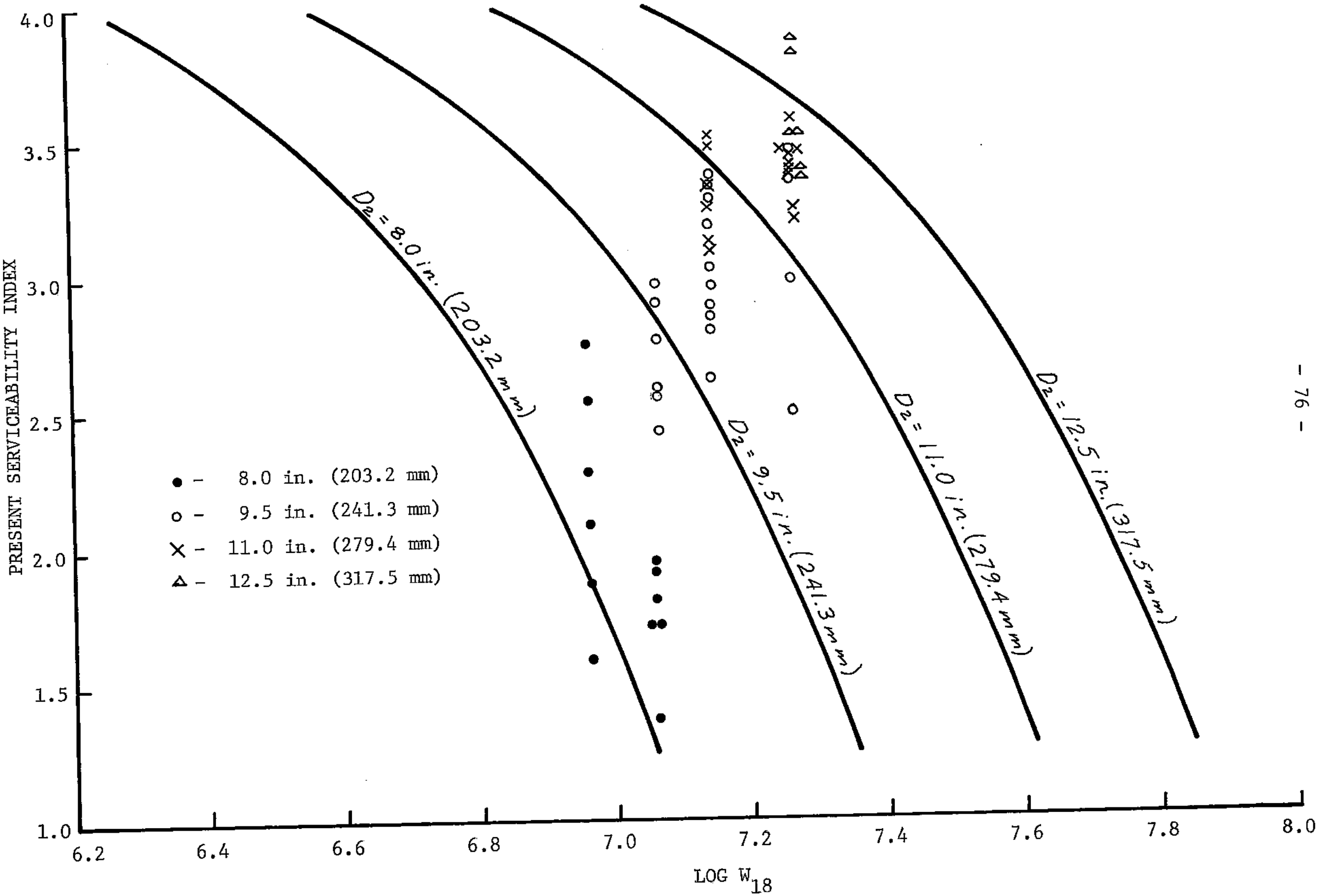


Figure 21, Performance curves using the modified equation in relation to the PSI of the original AASHO Sections in 1974.

granular material. The stabilized subbases (Figures 12, 13, 14), particularly the BAM, were outperforming the granular subbases.

Transverse cracking, D-cracking, and spalling at pavement edges, joints and cracks were the most prominent types of distress that developed in the experimental rigid pavement sections. Spalling and D-cracking were most severe at the transverse joints. Except for several cracks that formed late in the service life, there were no transverse cracks in the 15-ft pavement panels. In the longer panels, the number of cracks per panel increased as the panel length increased. Nevertheless, the overall amount of pavement distress per pavement mile was most strongly associated with the total number of pavement joints per mile rather than the amount of cracking that occurred. Joint lock-up, caused by dowel bar corrosion, was an important factor in the number of major (Class 3) cracks that formed. The reduction in major crack formation that was observed in pavement panels on the BAM subbases was associated with a greater capability to dissipate stress in the pavement panels on the BAM subbase than on the granular subbases.

D-cracking was found to be an important form of pavement distress because it developed in older pavements at joints and cracks where it increased the pavement roughness (RI). Spalling at joints and cracks also increased pavement roughness. Other forms of rigid pavement distress such as scaling, compression cracks and corner breaks were so minor that their effect on pavement behavior was not discernible.

The original flexible test sections that were included as part of the rehabilitated test road were all resurfaced. The unavailability of original AASHO sections for continued study precluded the application of the pavement serviceability-performance analysis in the study of the flexible pavement sections, but the new experimental secions that have been added have contributed new knowledge of flexible pavement behavior. The deepest ruts formed in the sections with crushed stone base while the shallowest ruts formed in the sections with CAM base. Rut development continued at a high rate in the granular base, but at the same time rutting in the stabilized bases developed at a moderate rate. Area cracking (alligator-type) was the dominant form of cracking in the granular base sections, but transverse and longitudinal cracking were the dominant form in the stabilized base sections. On the CAM base, many of the surface cracks were underlain by cracks in the base, which are believed to be caused by shrinkage. During freezing weather the cracks became tented, which added to pavement roughness. Overall cracking was least in the BAM base sections. The pavement sections with a BAM base were the smoothest riding. They actually became smoother as time progressed, although they tended to develop relatively deep ruts. The sections with a CAM base increased markedly in roughness with time, and had reached a terminal serviceability of 2.5 by 1974. Although the pavement sections on a granular base were intermediate in roughness between the sections with the CAM and the BAM bases, pavement distress (area cracking and rut depth) was most advanced, with a PSI of 2.9.

Wheelpath ruts developed in all flexible pavement designs. The track width between the front wheels of passenger cars is less than that of the heavy trucks, and the spacing of the rut depressions conforms to the track width of the trucks. As a result of the mismatch in track width, the passenger cars tended to wander in and out of the wheelpaths. The tendency to wander became a cause for motorists' concern, and they were seen stopped on the shoulders on numerous occasions examining the running gear for malfunction. The ruts were also a hazard in wet weather. Runoff water from rains accumulated in them in sufficient amounts to cause hazardous driving conditions (Figure 22). To avoid the hazard, motorists were forced to drive out of the wheelpaths (Figure 23).

Structurally, the pavements that had been resurfaced with bituminous overlays and the composite pavement sections were more than adequate for the anticipated service life. By the end of 1974, none of them had undergone a serious loss in serviceability. The most serious change was the formation of the ruts. The bituminous overlay containing the asbestos fiber rutted to a depth of 0.3 in. (8 mm) while those sections containing the other three fillers rutted to a depth of 0.49 in. (12 mm). Correspondingly, the asbestos fiber reduced rutting in the composite sections to 0.17 in. (4 mm) from 0.27 in. (7 mm).

The AASHO Road Test duplicate sections had a serviceability trend that was typical of the original AASHO test sections. The duplicate sections reached a terminal PSI of 2.5 before 1974, but they had carried more 18-kip ESAL applications than the comparable AASHO Road Test sections.

CONCLUSIONS

After 12 years of interstate highway service and the passage of almost 26 million vehicles as normal highway traffic, the experimental pavement sections in the rehabilitated AASHO Test Road have been resurfaced with bituminous concrete. The behavior of the test sections has been regularly documented during this period. Where applicable, the pavement serviceability-performance concept has been applied in the analysis of the data as a test of the performance equations and in the evaluation of the behavior of pavement designs, materials, and pavement structural elements that were included in the experimentation. From these studies the following conclusions seem to be justified:

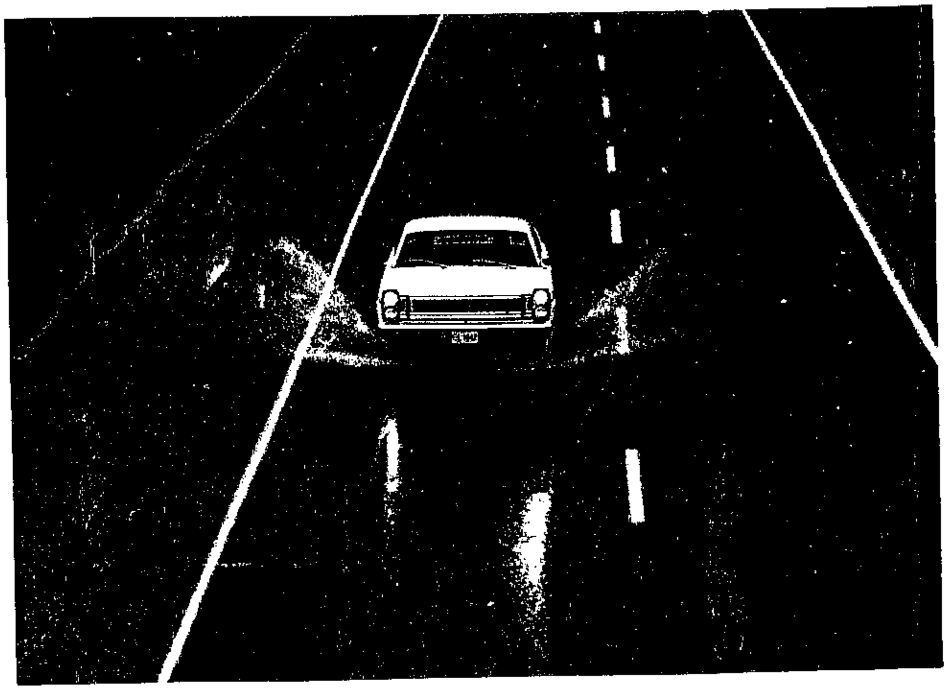


Figure 22. Picture of water in wheelpath ruts during rainstorm.

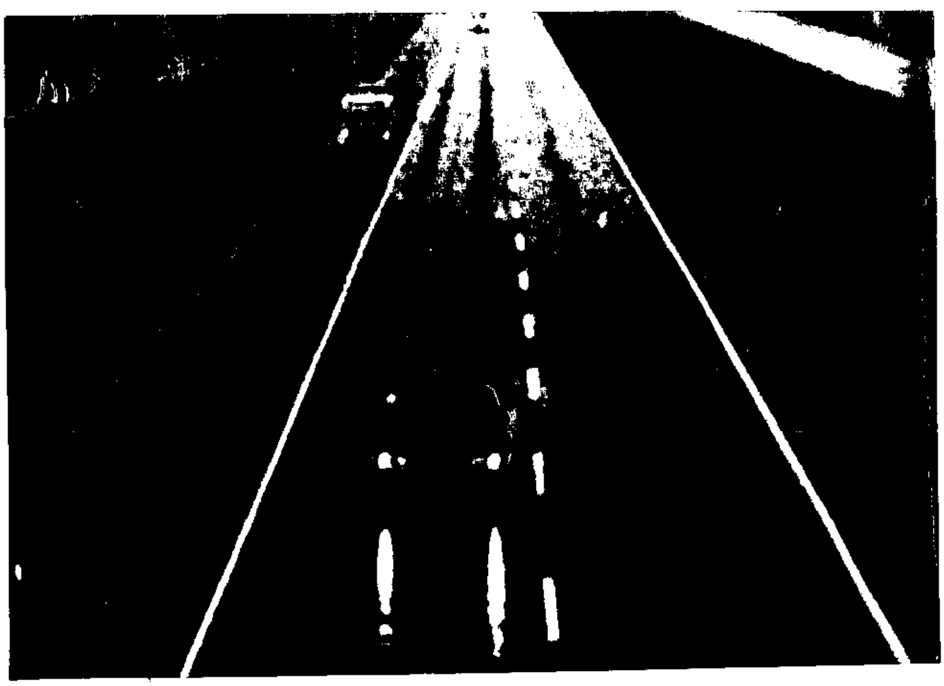


Figure 23. Picture of car avoiding water in wheelpath.

1. Pavement performance curves are useful in evaluating the behavior of pavement designs, materials and pavement structural elements such as bases and subbases.
2. The performance equation for rigid pavements predicted with reasonable accuracy the service life expectation of the 8.0-in. and the 9.5-in. rigid pavement sections in the original AASHO Test Road and the duplicate rigid sections that were built in 1962, but it tended to predict a greater expectancy for the original 11.0-in. and 12.5-in. rigid pavement sections than was actually observed.
3. The revised performance equation for rigid pavements fits the observed behavior of the 11.0-in. and 12.5-in. rigid pavement sections about as well as the original equation fitted the original data.
4. The revised equation increases the service life expectation of the 8.0-in. pavements slightly but reduces the expectation for the 9.5-in., 11.0-in. and 12.5-in. pavements by increasing amounts as slab thickness increases in comparison with the original equation.
5. With respect to performance of the rigid pavement sections, no difference in pavement behavior was observed that could be associated with the kind of granular subbase materials that were used.
6. The stabilized-aggregate subbases (BAM and CAM) were effective in improving the performance of rigid pavements over those on granular materials.
7. The improved performances associated with the BAM subbase was ascribed to a difference in subbase drag, which resulted in fewer

transverse cracks (8) and more uniform winter joint openings, particularly in pavements with 100-ft pavement panels.

8. The most important types of rigid pavement distress with respect to pavement performance were transverse cracking, D-cracking, and spalling at the pavement joints and cracks.
9. Corner breaks and compression cracks occurred only in minor amounts.
10. Pavement pumping occurred only in minor amounts.
11. Only one blowup occurred in the mainline pavement during the study period, and it was not in an experimental pavement section.
12. The flexible pavement sections on a CAM base had the highest RI, and those on a BAM base had the lowest RI.
13. The flexible pavements on a CAM base developed higher RI's earlier in their service life than those on the crushed stone or the BAM bases.
14. The roughness of the CAM base was associated with tenting of the transverse cracks in the pavement surface.
15. Feathered construction joints in the CAM bases were associated with humps in the pavement surface.
16. The rate of rut formation in the flexible pavements tended to decrease with time on the BAM and CAM bases but continued at a high rate on the granular bases.
17. The resurfaced flexible AASHO pavement sections proved to be structurally more than adequate for the interstate highway traffic during the observation period. However, the ruts that formed in the wheelpaths were approximately one-half inch deep.

18. On the granular base course, the rutting was confined to the surface course and the base course. It was not apparent in the subbase.
19. Rut depths were reduced on the portland cement concrete base on the CAM base.
20. Replacement of part of the mineral filler with asbestos fiber in the surface course mixtures reduced the depth of rutting by approximately one-third.

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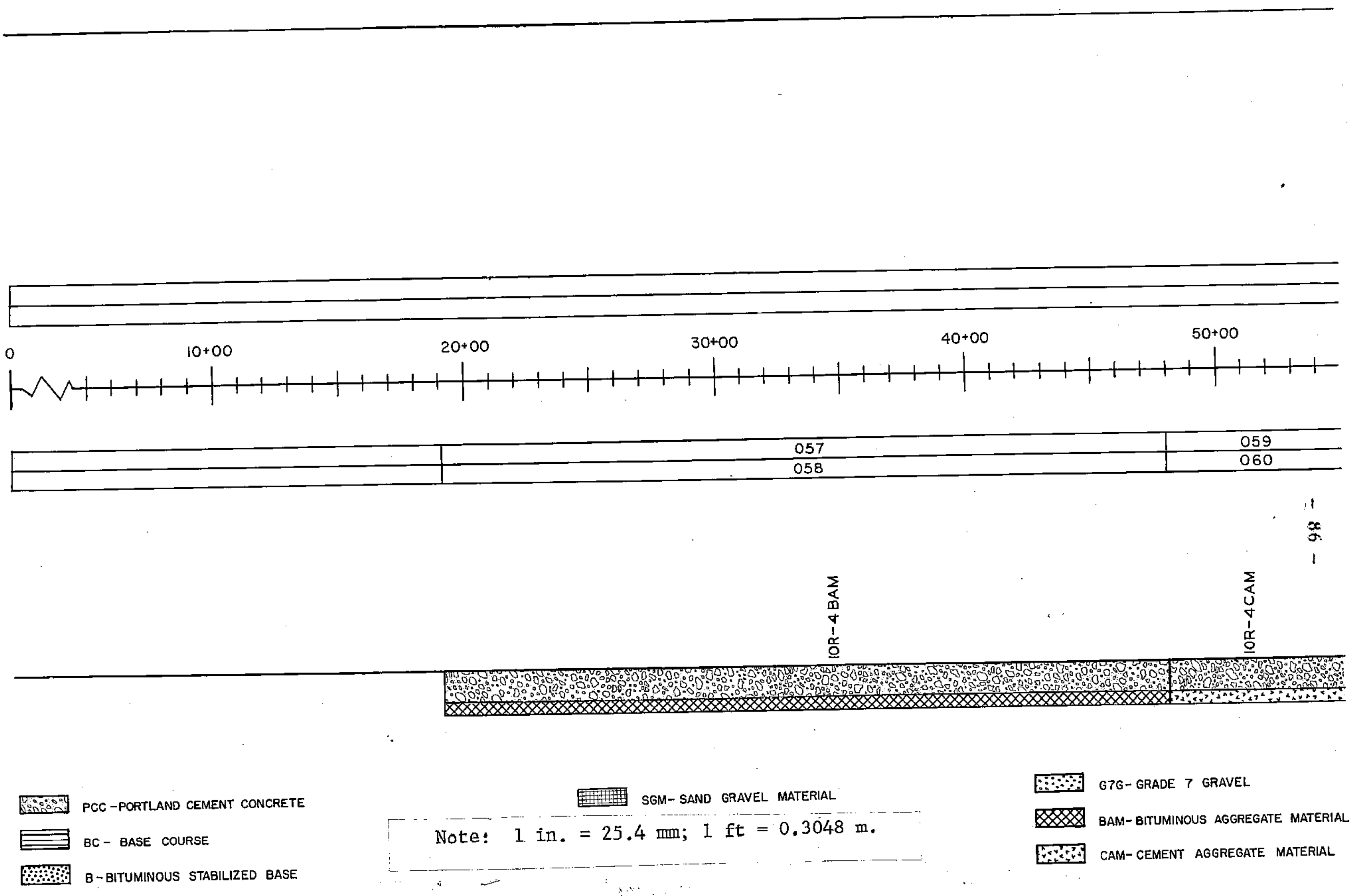
APPENDIX

A

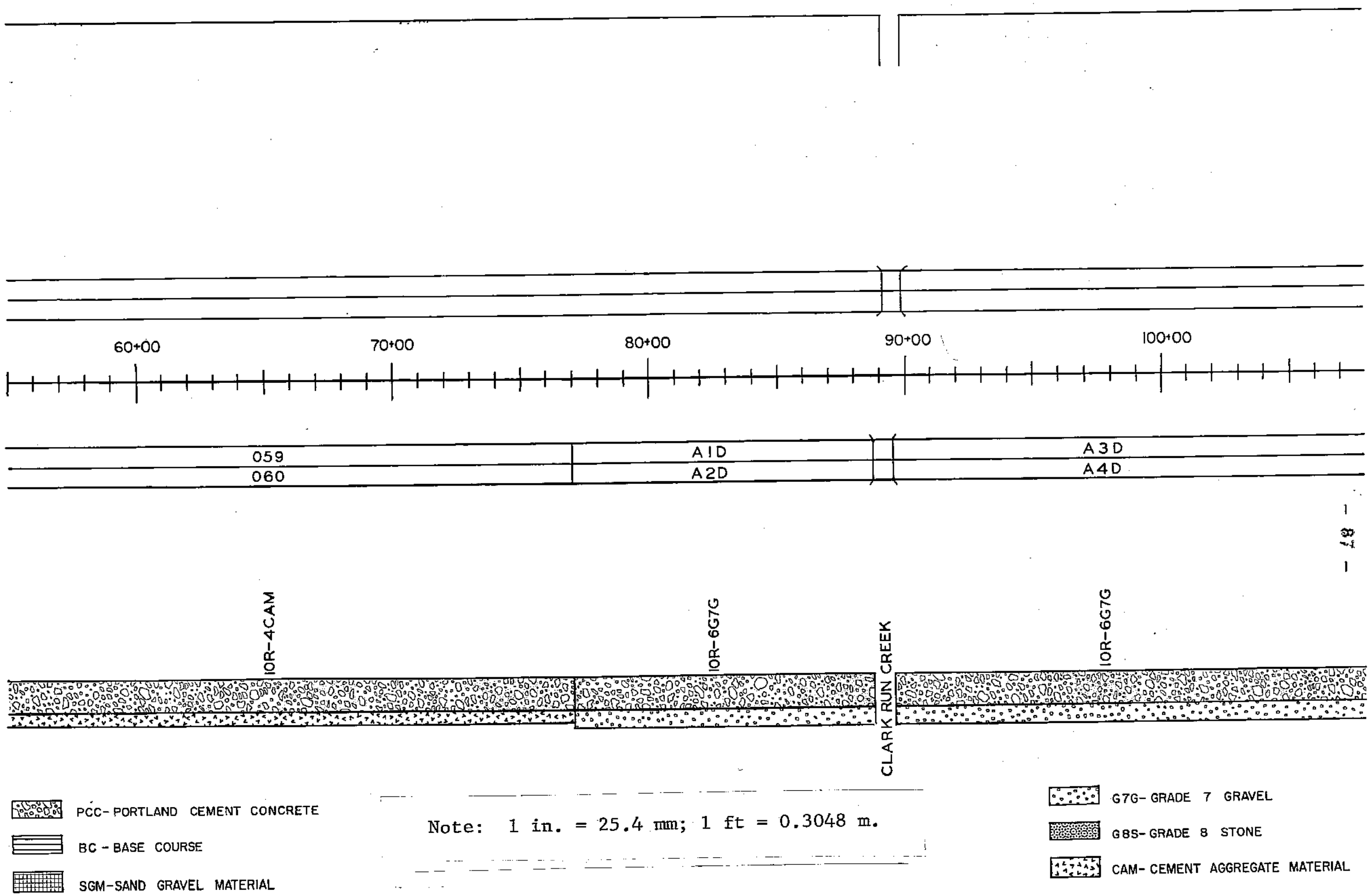
Layout of Test Sections

In Figure A-1 and A-2, the symbol identified as BC-Base Course is the crushed stone-special which was the original base material in the AASHO Test Road. Test sections 016 and 018, which have the same symbol, represent salvaged crushed stone-special placed when the pavement was rehabilitated.



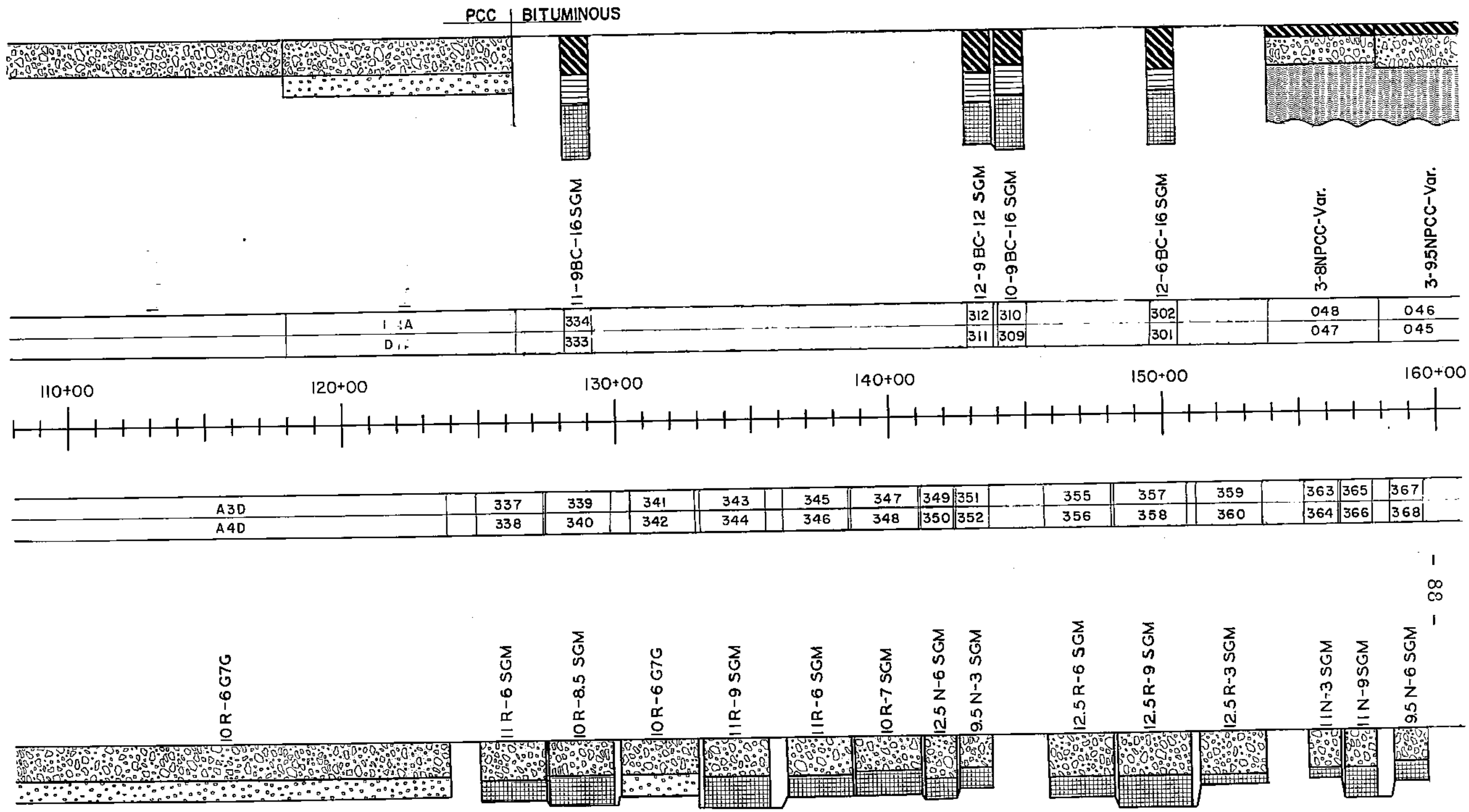
Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m.

Figure A-1. Layout of test sections.



Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m.

Figure A-1. Layout of test sections. (continued)



PCC-PORTLAND CEMENT CONCRETE

I-II-BITUMINOUS CONCRETE

BC-BASE COURSE

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m.

SGM-SAND GRAVEL MATERIAL

G7G-GRADE 7 GRAVEL

VAR-VARIOUS MATERIAL

Figure A-1. Layout of test sections. (continued)

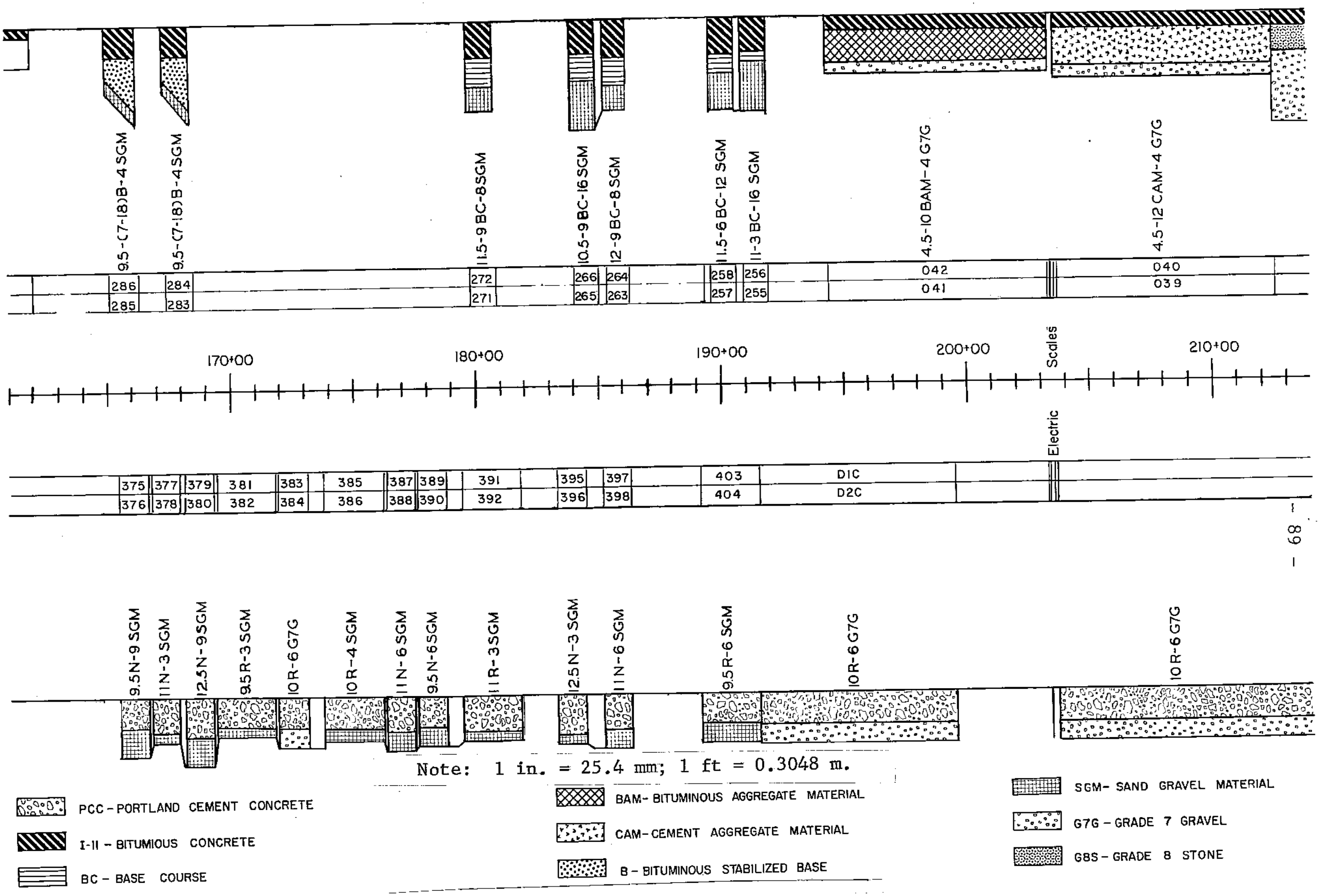
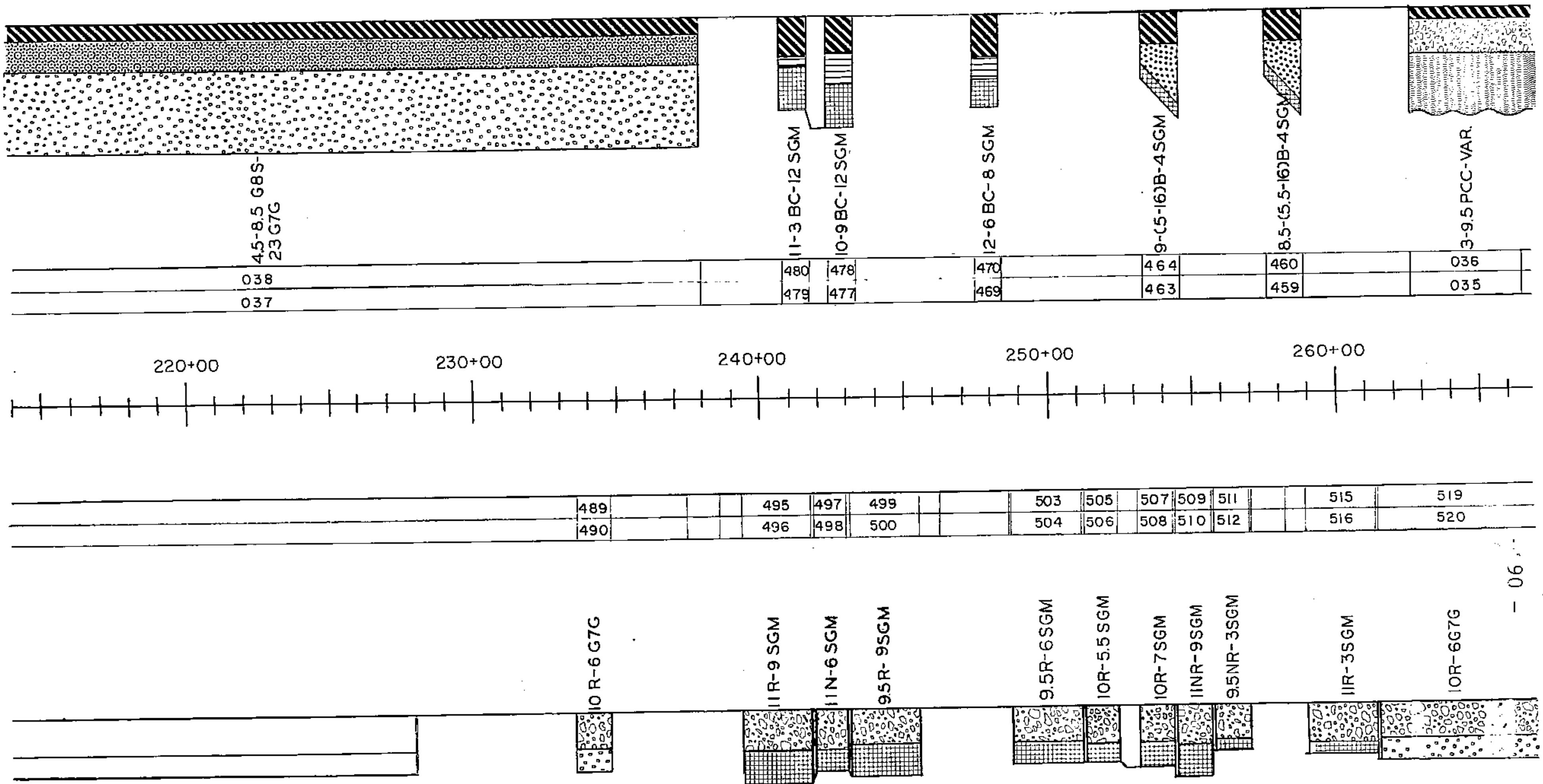


Figure A-1. Layout of test sections.
(continued)



Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m.

PCC- PORTLAND CEMENT CONCRETE

I-II - BITUMINOUS CONCRETE

BC- BASE COURSE

B - BITUMINOUS STABILIZED BASE

SGM- SAND GRAVEL MATERIAL

VAR- VARIOUS MATERIAL

G7G- GRADE 7 GRAVEL

G8S- GRADE 8 STONE

Figure A-1. Layout of test sections.
(continued)

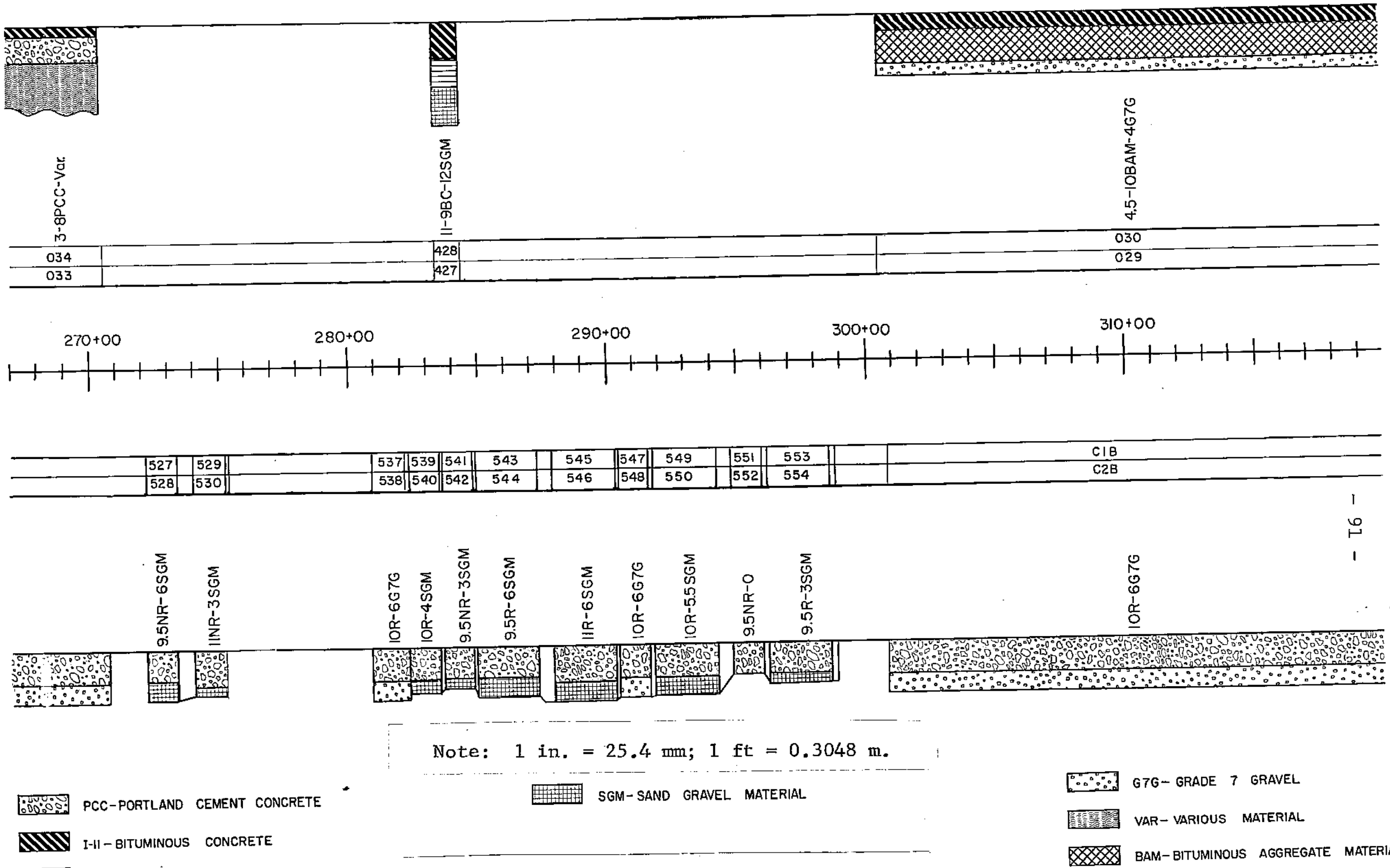
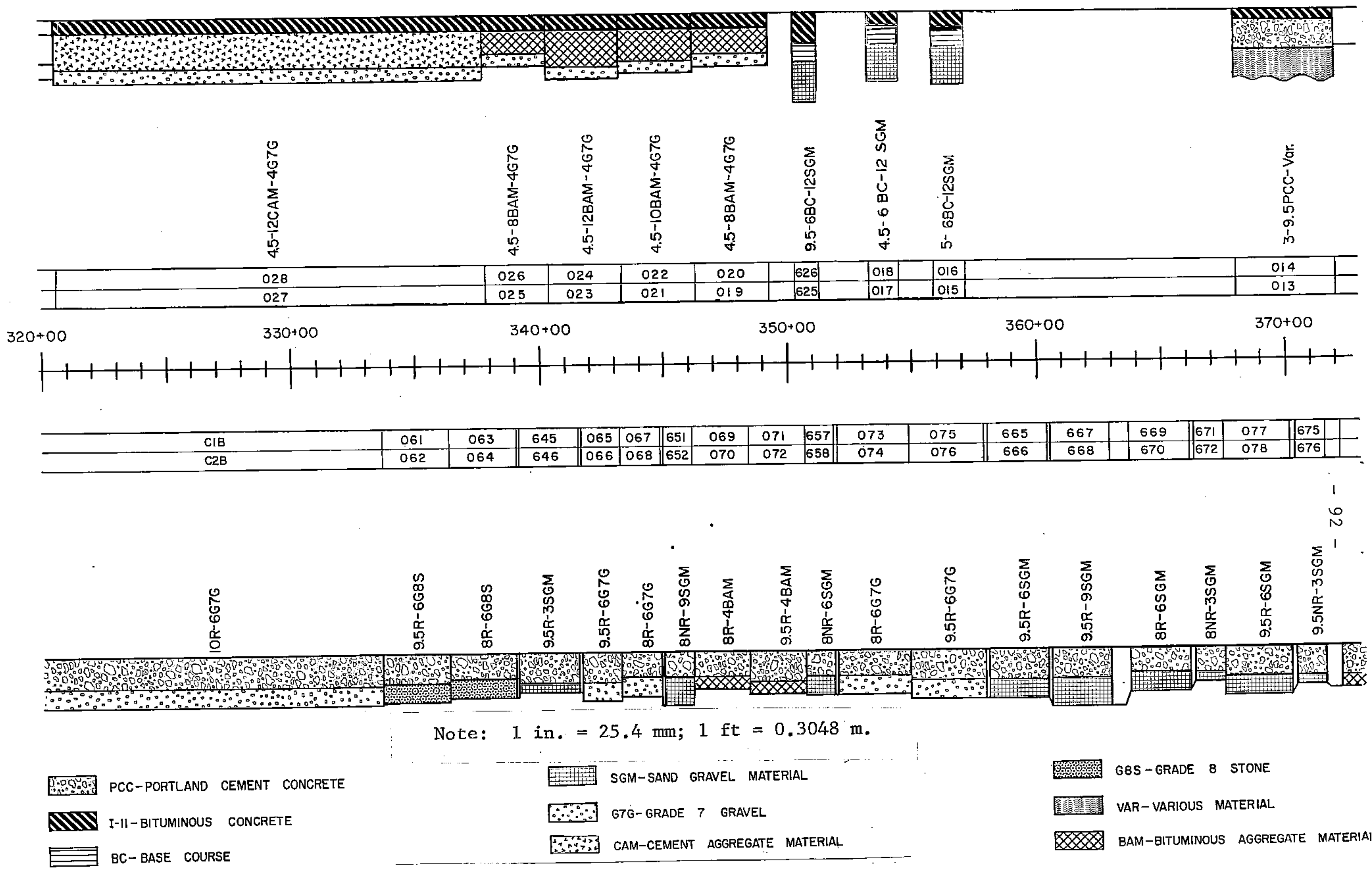
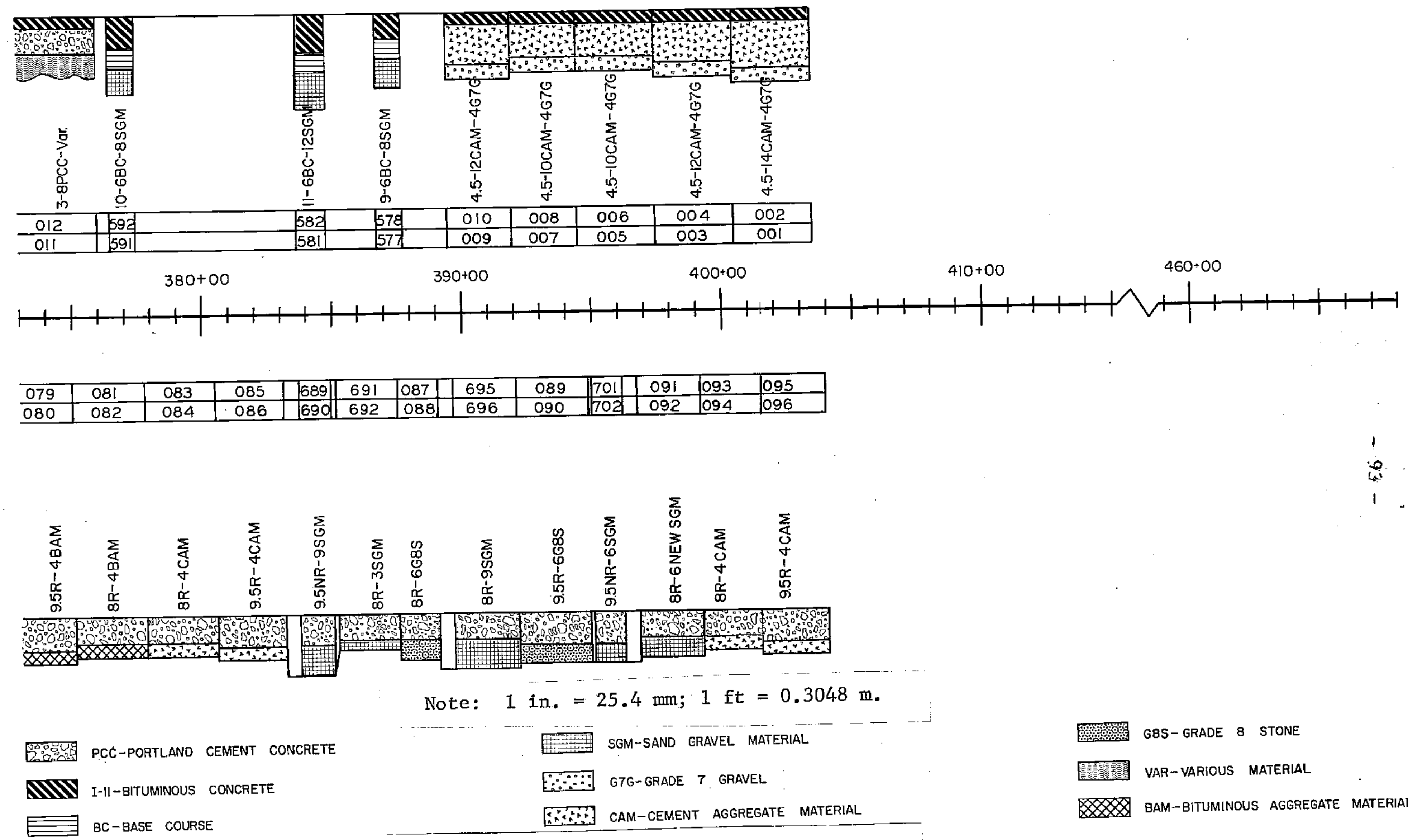


Figure A-1. Layout of test sections.
(continued)



Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m.

Figure A-1. Layout of test sections.
(continued)



Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m.

Figure A-1. Layout of test sections. (continued)

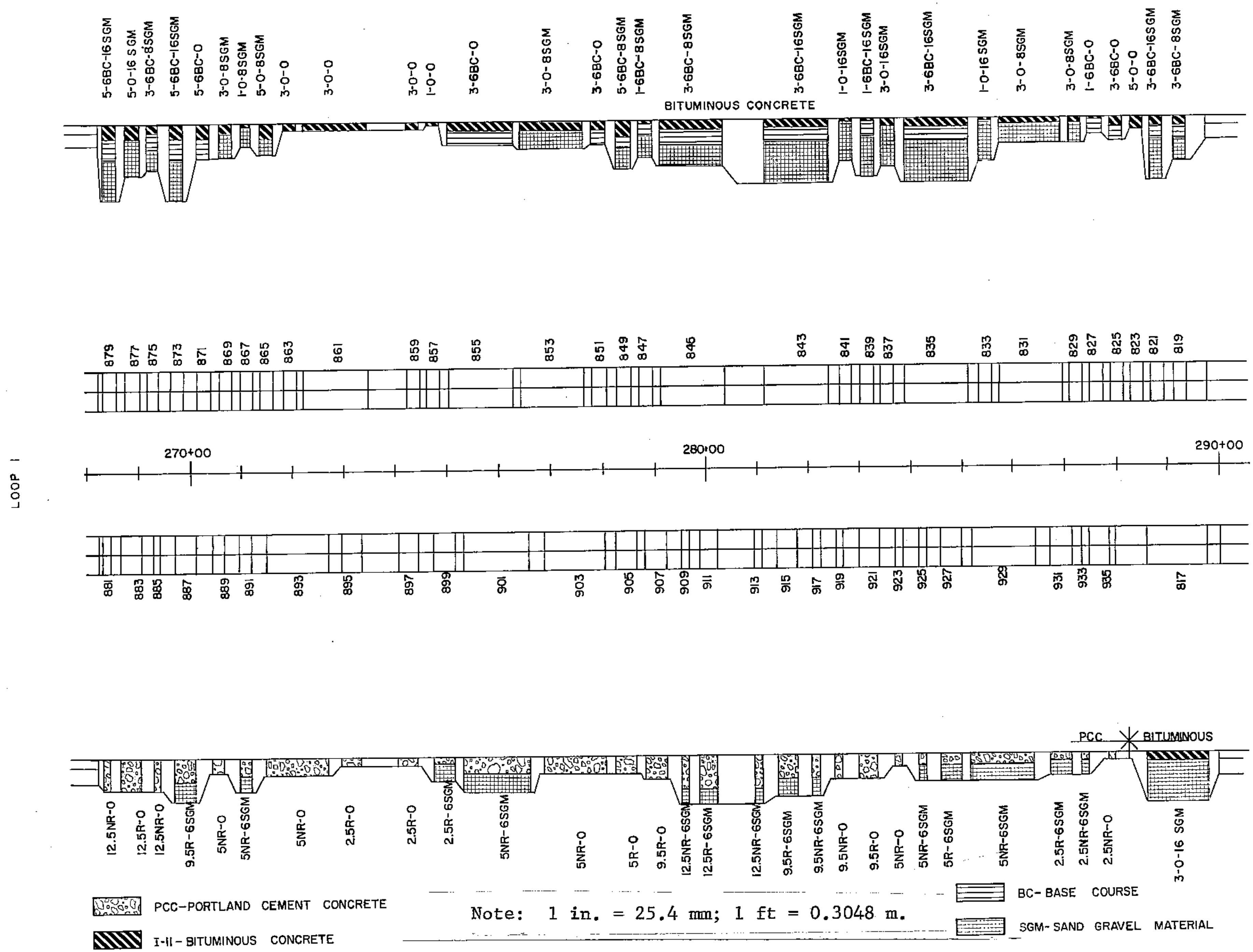


Figure A-2. Layout of test sections - Loop 1.